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A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

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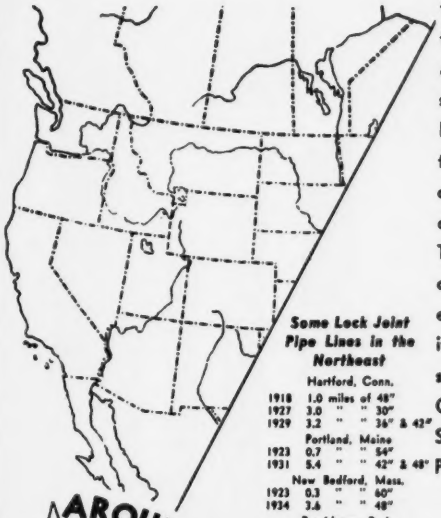
This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September on such date as the Executive Committee may designate.



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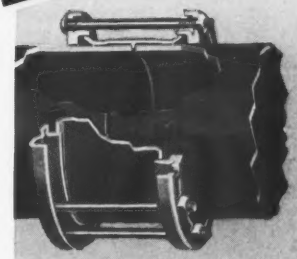

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New England Water Works Association
ORGANIZED 1882.

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This Association, as a body, is not responsible for the statements or opinions of any of its members

SILVICULTURAL AIDS IN THE CONTROL OF THE GYPSY MOTH IN WATERSHED FORESTS.*

BY A. C. CLINE†

The gypsy moth (*Porthetria dispar* L.) was accidentally introduced from Europe into New England, in the vicinity of Boston, in 1868, and for nearly fifty years it has been one of our most important forest and shade tree pests, occasioning the expenditure of many millions of dollars by public and private agencies in an effort to control its spread.

Damage to trees is caused by the feeding of the larvae (caterpillars) on the foliage. Thrifty trees of the broad-leaved (hardwood) species are never killed by a single defoliation, but the loss in vigor caused by repeated stripping may pave the way for other pests, and lead eventually to a considerable mortality. With conifers, however, (with the exception of larch) a single complete defoliation proves fatal, since they lack the deciduous character of the broad-leaved species and the ability to refoliate following the loss of their leaves. I recognize that in watershed forests there are other than timber or esthetic values involved, and that large colonies of insects in the caterpillar stage are highly undesirable irrespective of any actual damage to the trees or the mere unsightliness of bare branches during a few weeks in early summer.

When first hatched, the caterpillars are extemely small and, with the aid of a fine silken thread which they spin, they may be blown long distances with the wind, thus providing a means of wide dispersion in certain directions. In Figure 1 it will be noted that the farthest spread from the initial point of establishment has been toward the northeast, presumably due to the prevalence of westerly winds during the hatching season. Dispersion is also made possible by vehicular traffic on the highways, by railroads, the shipment of nursery stock, ornamental trees and any materials on which eggs or caterpillars may be carried, as well as by the slower method of normal spread from the edges of heavily infested

*In the preparation of this paper the author has borrowed freely from the following publications: Silvicultural Control of the Gypsy Moth, by C. Edward Behre, A. C. Cline and W. L. Baker, Bull. 157, Mass. Forest and Park Assoc., 1936; and A Study of the Gypsy Moth in the Town of Petersham, Mass., in 1935, by W. L. Baker and A. C. Cline, Jour. of Forestry, August 1936.

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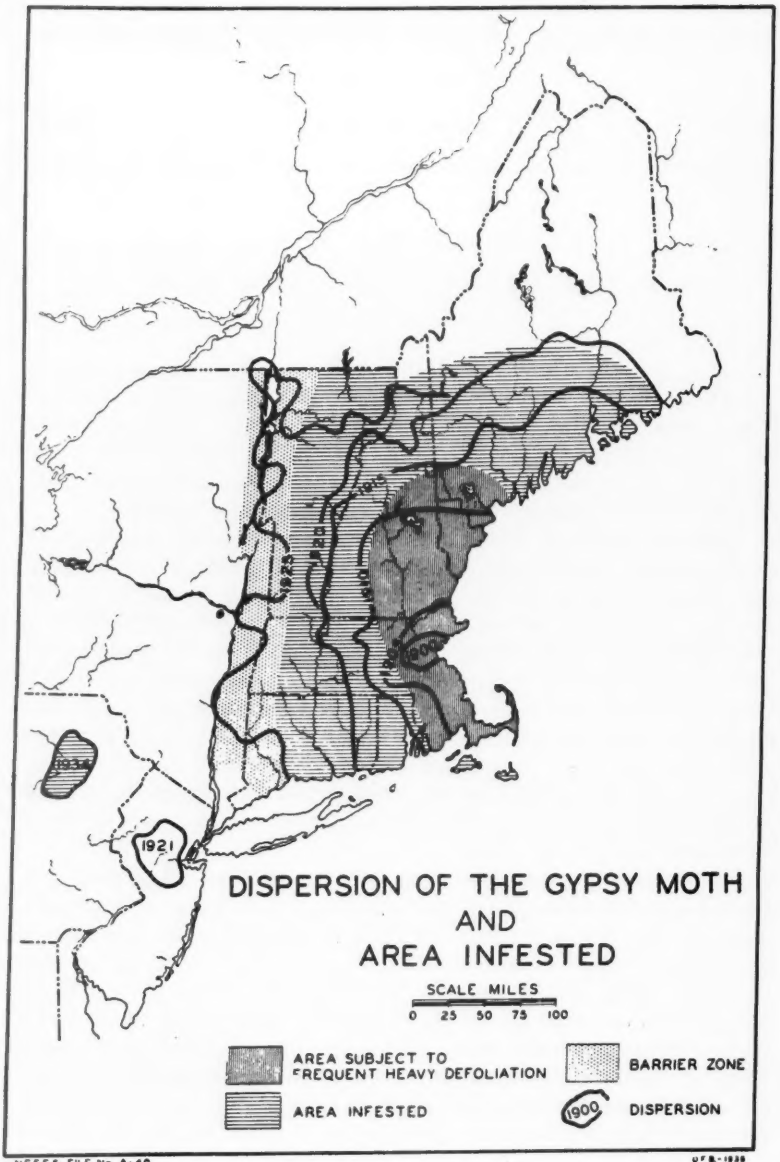

Figure 1

FIG. 1.

areas. The latter takes the form of short migrations by the larger caterpillars crawling in search of food, after the trees on which they spent their earlier lives have been defoliated. Fortunately, the female moth is unable to fly; otherwise, spread would be even more rapid.

Extent of Infestation. As with all forest insects, the extent of infestation by the gypsy moth in a given year has varied widely with the character and distribution of its food supply, climate, natural enemies and numerous other factors, which taken together cause the irregular ups and downs in moth populations, commonly referred to as cycles. Cycles may be thought of as being both regional and local. Unfavorable climatic conditions regional in extent, such as a winter sufficiently cold (about 25° F. below zero) to kill egg masses that are not beneath the snow or otherwise protected, or an excessively dry spell at the time when the caterpillars are just emerging and need moisture to sustain life until food is found, may cause a region-wide decline in moth populations requiring a succession of more favorable seasons for rebuilding to the former abundance. On the other hand, a single colony in a given locality may be cyclic because at its peak there is not enough edible food to go around; thousands of caterpillars die from starvation or the "wilt" disease, and several years may be required for the colony again to build up to a condition of congestion and inadequate food supply.

All stands within an infested region are not equally susceptible to attack. Even within localities rated as most heavily infested, individual trees and whole stands may repeatedly escape defoliation. The reason for this is found in the discriminating feeding habits of the caterpillars, which show a decided preference for the foliage of certain species of trees.* Among these highly favored food species the most important and common are the oaks, gray birch and poplar. Others are apple, alder, basswood, willow and hawthorn.

Many of you undoubtedly have observed the severity of attack on Cape Cod, where oak is so predominant, and farther inland where stands of oak, gray birch and poplar, either pure or mixed, occur interspersed with other types of forest growth. In this connection an explanation of the increasingly heavy infestations in recent years in central Massachusetts lies in the enormous increase in these highly favored food trees following the clear-cutting of thousands of acres of second growth white pine during the past thirty years. For it so happens that "old field" white pine is nearly always followed by hardwoods in which the oaks, gray birch and poplar are prominent species.

On the other extreme, the list of least favored species, some of them scarcely ever fed upon, includes ash, locust, tulip poplar, black walnut, red cedar and balsam fir, while those occupying an intermediate place

*For complete list of tree species arranged according to preference see "Silvicultural Control of the Gypsy Moth" by C. Edward Behre, A. C. Cline and W. L. Baker, Bull. 157 of the Mass. Forest and Park Assoc., 1936.

as host species comprise the maples, birches (except gray birch), hickories, elm, black cherry and hornbeam. The leaves of these last-named are edible, but are not favored, and stands composed of such species will not support a heavy infestation. In addition to these three classes of food plants, there is a group made up chiefly of the pines, spruces, hemlock and beech, which are definitely unfavorable as food for the young caterpillars but favorable for the older ones.

Since we cannot control the climate, and forest entomologists inform us that the introduction and propagation of parasitic enemies is not expected to prevent further epidemic outbreaks from time to time, it remains that by far the most promising method of indirect or natural control is through altering the forests themselves, that is, by reducing the abundance of the highly favored food species. And such a policy is greatly strengthened by the fact that most of the highly favored food species and many of the highly susceptible stands are undesirable from a forestry standpoint—either inferior species or slow-growing, low grade stands that should be replaced by something of greater economic value.

With few exceptions, the fine forests that the first white settlers found here contained much smaller percentages of favored food species than the secondary types of growth that have followed as a result of farm abandonment, clear-cutting or fires. The more exacting hardwood species, many of them resistant to gypsy moth attack, which require shade and moisture for successful growth have given way to light-demanding species, usually of short-lived or inferior character, but capable of growing under conditions of full exposure and comparative dryness. And, with the repeated clear-cutting of second growth stands, the growing stock becomes progressively deteriorated, the proportion of highly favored food trees is increased and with it the susceptibility of the stand to defoliation. So you see that much of our present trouble is due to the neglect of our forests, of failure to hold the weeds in check and to encourage the development of the better trees. Indeed, it may be said that our forests are like gardens that have been allowed to go to weeds.

Silvicultural Control. Silvicultural control in the form of such operations as weedings, improvement cuttings and thinnings, may at present supplement and perhaps in time wholly replace the direct or artificial methods of controlling the gypsy moth. Creosoting egg masses, banding trees and spraying foliage, now so commonly practiced, is most assuredly an extremely costly effort to control this pest, which is established over a large territory and shows no signs of decrease. On the contrary, infestation reached a new peak in central Massachusetts in 1935, and recently new gypsy moth colonies were discovered near the Hudson River in New York State. Not only are these artificial control measures so expensive when effectively applied that their cost would soon exceed the forest values they aim to protect, but at best they are only palliative. They are designed

to reduce infestations as they arise, without respect to the biological conditions that largely control the development and spread of moth populations. They do not get at the source of the trouble, which is the food supply. Reason compels us now to look upon the gypsy moth in New England as a native insect, like the tent caterpillar or the white pine weevil; it is futile to attempt its extermination. All we can and should hope for is a reasonable degree of control at a reasonable cost.

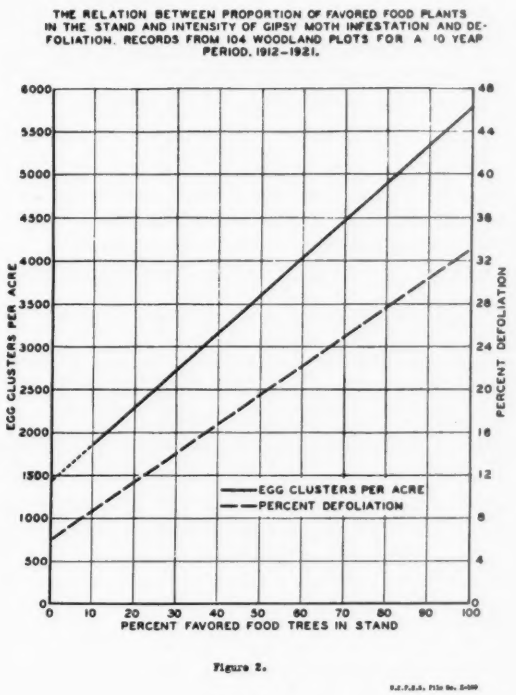
THE RELATION BETWEEN PROPORTION OF FAVORED FOOD PLANTS IN THE STAND AND INTENSITY OF GIPSY MOTH INFESTATION AND DEFOLIATION. RECORDS FROM 104 WOODLAND PLOTS FOR A 10 YEAR PERIOD, 1912–1921.
EGG CLUSTERS PER ACRE
PERCENT DEFOLIATION
PERCENT FAVORED FOOD TREES IN STAND
EGG CLUSTERS PER ACRE
PERCENT DEFOLIATION
Figure 2.

FIG. 2.

In order to bring some factual information to bear on my arguments for silvicultural control, I wish to show a graph which brings out the relation of defoliation to the character of the stand in respect to the proportion of favored food trees present (Fig. 2). This is based on 104 woodland sample plots established by the U. S. Bureau of Entomology and Plant Quarantine in various parts of the infested region. You will observe a direct relationship, that is, the higher the proportion of favored food trees in the stand, the greater the per cent. of the foliage volume consumed by the caterpillars.

I wish to present also a table (Table 1) based on a study of the gypsy moth in the town of Petersham, in 1935*, which shows that in every one of 81 cases where stands were completely defoliated highly favored food species comprised over 50 per cent. of the stand, and in more than one-half

TABLE 1.—OCCURRENCE OF CERTAIN TREE SPECIES IN HEAVILY DEFOLIATED AREAS IN THE TOWN OF PETERSHAM, 1935.

SPECIES OR COMBINATIONS OF SPECIES IN STAND.	NUMBER OF CASES WHERE SPECIES COMPRISED INDICATED PERCENTAGE RANGE OF STAND.		
	50–75 Per Cent.	75–100 Per Cent.	50–100 Per Cent.
White oak	0	0	0
Red oak	2	0	2
Poplar (2 species)	8	4	12
Gray birch	13	21	34
Gray birch and poplar	18	39	57
Gray birch, oaks, and poplar	20	56	76
Gray birch, oaks, poplar, and alder	..	..	81

TABLE 2.—COMPARISON OF DEFOLIATED STANDS WITH THE MARGINS OF ADJOINING STANDS.

Plot No.	AVE. PERCENTAGE OF FAVORED FOODS.		AVERAGE PERCENTAGE OF DEFOLIATION.*					
			Favored Foods.		Unfavored Food.		All Trees.	
	Defoliated Area	Margin of Adjoining Stand.	Defoliated Area.	Margin of Adjoining Stand.*	Defoliated Area.	Margin of Adjoining Stand.	Defoliated Area.	Margin of Adjoining Stand.
11	86	12	87	33	37	5	80	9
16	86	21	91	91	58	11	86	28
24	87	15	100	24	54	6	94	8
38	92	19	100	10	47	0	96	2
39	95	1	100	0	100	0	100	0
40	97	32	85	44	75	6	84	18
41B(NE)	91	10	99	36	50	17	95	19
41B(W)	91	16	99	37	50	15	95	17
42	90	51	96	26	34	3	90	14
51	91	35	100	14	72	9	98	11
71	83	5	100	100	54	11	92	15
72	87	15	87	8	55	5	83	6
Mean	89.7	19.3	95.3	35.3	57.2	7.3	91.1	12.2

*The per cent. of the total foliage volume of the stand consumed by caterpillars.

the cases over 75 per cent. of the stand. These defoliated areas, occupied mostly by oak, poplar and gray birch, pure or in mixture, do not cover wide stretches in Petersham, but are scattered about, a few acres here and there, usually very definitely associated with past farming or logging practices, or with areas of poor soil on ridge tops or other unfavorable

*A Study of the Gypsy Moth in the Town of Petersham, Mass., in 1935, by W. L. Baker and A. C. Cline, Jour. of Forestry, August, 1936.

sites. Thus heavy infestation was not general throughout the forested sections of the Town, but was strictly limited to those stands composed very largely of highly favored host species. This is further evidenced by Table 2, which shows a comparison of heavily defoliated stands with the margins of adjoining stands of different species composition. Invariably it was observed that the heavy stripping ceased abruptly where highly favored species gave way to unfavored species.

Recommended Methods of Control. Now to come to the methods of silvicultural control that I should like to recommend. First let us consider coniferous plantations. Fortunately, newly hatched caterpillars cannot feed on pine or spruce. They must succeed in reaching one of the favored hardwood host species or larch in order to survive. It is only after they have passed the second molt, at which time they have attained a length of about ½ in., that they may attack these two commonly planted conifers. Unless hardwoods of favored food species have been permitted to grow up in the plantation, the only hazard lies in adjoining hardwood stands containing high proportions of host species. Reducing the proportion of such species to below 50 per cent. of the total in a marginal strip about 100 ft. wide will ordinarily prevent defoliation of the planted conifers to any serious degree. If the adjoining stand should happen to be made up wholly of highly favored food trees, it will be necessary to clear a protective strip of similar width, which later may be extended to include the entire area of highly susceptible growth. Such clearings may then be safely planted with conifers or with mixtures of conifers and unfavored hardwoods.

Similarly, natural stands of conifers may be protected. Oftentimes, on abandoned fields and pastures, or on light sandy soils, pine is found growing underneath or among gray birch, poplar and oak. Security demands the cutting of such hardwoods when immediately overtopping the pine, or of thinning or cutting a protective strip in exactly the same manner as in safeguarding a plantation. Such a treatment ordinarily will take the form of what is known as a release cutting, often practiced by landowners in the case of pine overtopped by gray birch on abandoned farms, and one of the most profitable silvicultural treatments in common use.

Mixed stands of conifers and hardwoods lend themselves to protective treatments in the form of thinnings or group cuttings to reduce the proportion of highly favored food species to safe limits. Here the chief concern is to prevent the stripping of the conifers. Reduction of highly favored hardwood foliage to at least one-half the foliage volume of the conifers will in most cases assure adequate protection. In highly susceptible stands thinning may have to be supplemented by group cuttings of highly favored food trees, the openings so made being later planted to conifers or resistant hardwood species.

In hardwood or conifer-hardwood mixtures containing high proportions of such resistant species as ash, maple, yellow and black birch, tulip poplar or black cherry, little or no protective cutting will be needed. Several of these are exacting species that occur abundantly only on the better soils, which explains the comparative freedom of the better quality mixed hardwood stands from gypsy moth attack. The Berkshire region and extensive areas in north-central New England support stands of this character.

In southern New England pure stands of oak, highly susceptible to attack, are of common occurrence on light soils and exposed sites. Fre-

Harvard Forest Photograph.

FIG. 3.—A STAND OF HIGHLY FAVORED FOOD SPECIES, POPLAR, ADJOINING A RED PINE PLANTATION.

Cutting the poplar will both safeguard the red pine from stripping and save marginal pines now being suppressed. (See arrows.)

quently, they are of stump sprout origin, slow-growing, scrubby, and of no value other than for cordwood. Such stands might well be clear cut and the land planted to conifers. Oak stands of good quality, on the other hand, are not to be so drastically treated, but should gradually be altered by way of encouraging any available unfavored hardwood species, even to the extent of including species of second-rate commercial importance, such as red maple and black birch. Frequently, unfavored species occur in the understory, and they in time may be given a dominant place in the stand by a judicious cutting of some of the poorest oaks in the

overstory. Otherwise, small group clearings may be made over a period of years, and these openings planted to resistant hardwood species. The periodic application of group cuttings will eventually serve to convert the stand into a relatively immune group selection (all-aged) form in which oak will be limited to somewhat less than 50 per cent. of the total. More or less natural reproduction of resistant species generally becomes established in group clearings, thus permitting some saving in the cost of planting. All areas of young growth, whether planted trees or natural reproduction in small openings or on extensive clear cuttings, must be weeded in order to free the most desirable stems from overtopping stump

Harvard Forest Photograph

FIG. 4.—A MIXED STAND OF WHITE PINE AND HARDWOODS ON CUTOVER LAND.

Cutting out gray birch and rank-growing red oak stump sprouts improves the stand and at the same time reduces the gypsy moth hazard. Both are forest weeds and also highly favored food species.

sprouts and other inferior elements. In developing high quality mixed hardwood stands from small sapling growth, as many as two or three weedings may profitably be carried out over a period of ten to fifteen years. In these, no attempt should be made to eliminate the oak, but only to prevent its becoming the dominant species in the young stand.

In the aggregate, a considerable area in New England supports stands entirely or predominantly of inferior species, such as gray birch, poplar, and scrub oak—all highly susceptible to gypsy moth attack. On watershed forests, as well as on any others, good management demands the complete

removal of such worthless growing stock and its replacement with stands of prospectively high value. Clear-cutting followed by the planting of conifers, unfavored hardwoods, or mixtures of both, is plainly the one and only silvicultural treatment that will serve to control the gypsy moth and at the same time to restore the land to productive use.

Conclusion. In conclusion, let me say that I am not recommending the immediate abandonment of the direct methods of control, through creosoting or spraying, in favor of the indirect methods, but rather a gradual transition from the one to the other. Alteration of the forest composition may proceed slowly or rapidly, depending upon the character of the growing stock, as well as upon the availability of funds for silvicultural work. It is plainly seen, however, that as the proportion of unfavored host species is increased, the need for direct methods of control is correspondingly lowered, until the time finally is reached when the forest growing stock may be said to be "moth proof." Lastly, I wish to emphasize that the elimination or reduction of the highly favored food species will generally conform to desirable silvicultural practices from the timber crop production standpoint. Under New England conditions silvicultural control has this added advantage of serving the objectives of forest improvement, with little of no extra expense directly chargeable to insect pest control.*

*Members of the New England Water Works Association may obtain upon request and without charge copies of the bulletin 'Silvicultural Control of the Gypsy Moth" by C. Edward Behre, A. C. Cline and W. L. Paker, published by the Massachusetts Forest and Park Association, 1936. Requests should be addressed to Mr. Harris A. Reynolds, Secretary, Massachusetts Forest and Park Association, 3 Joy Street, Boston, Mass.

SIZES OF METERS FOR SERVICES.

BY ROGER W. ESTY*.

[*Read April 23, 1937.*]

What are the factors that determine the proper size of meter that should be set in a pipe line. While I am not an authority on this matter,—for there are many water works men far better qualified to discuss this subject than I am—I have been asked to present my viewpoint on this matter and explain why I use perhaps an 1½-in. meter on a 4-in. line, or perhaps a 4-in. meter on a 6-in. line.

The proper sizing of meters is a very interesting subject, as each customer presents a different problem. We say that theoretically a certain size of meter should go on a particular size service, but practically it may not be the proper meter. What then does help to determine the proper size of meter. I think that the word "experience" explains more completely why we do this or that. Experience in determining in your own mind the proper size of meter that should be selected plus a 24-hour flow test under actual conditions on a recording chart will, in my opinion, leave practically no doubt as to what size should be selected.

Getting back to theory: I think that is a word which a great many times is misunderstood and is very misleading. We are told that for a certain flow of water under a known pressure through a given length of pipe we should have a certain size of pipe, and on or in this pipe line there should be a certain size of meter. In a great many instances this does work to perfection, but in other cases it just doesn't seem to work out.

Take the practical man backed up with a lot of experience, he has what you may call another sense, that is meter sense, an uncanny way of knowing the kind and size of meter best adapted for a particular service. It is by experience that we learn that the demands of a laundry are different from those of a shoe factory, and those of a leather factory different from those of a lamp factory. From these experiences we obtain knowledge of what is the best type of meter and the size best adapted to varying types of conditions that are produced by these different demands.

We have made many changes in the kind or type meter that we feel should be set in a line, and the size that that particular customer requires. As an illustration, in my opinion a laundry or irrigation line requires a velocity type meter; a school building, shoe factory, etc., a displacement meter, better known as a disc meter. A leather factory or lamp factory should have a combination velocity and displacement meter, better known as a compound meter.

*Superintendent, Water Board, Danvers, Mass.

In the first case, the laundry or irrigation services are either using large volumes of water or none at all. In the second case, in schools and shoe factories the displacement meter should be used as their demands are more nearly small uniform and constant flows. In the third case, the demands of a leather or lamp factory sometimes are high, then they drop off to a very small demand. In a leather factory the demands are sometimes 500 to 600 g.p.m., then they may drop to 5 to 20 g.p.m.

Then too, on many of these 4-, 6- and 8-in. lines there are fire hydrants; that demand has to be taken into consideration. We have a group of institution buildings supplied by an 8-in. line with a 6-in. meter in the line. According to our chart readings, the demand never exceeds 40 g.p.m. and the night rate is only 5 to 10 g.p.m. Therefore, in this case a 1-in. meter would suffice, but it is necessary to have a 6-in. meter for the fire-protection demands. In such cases our rates are so devised that the service has to pay for the demand in the form of a service charge. The 1-in. meter service charge would be $2.80 per quarter, and the 6-in.charge is $42.40 per quarter. The service pays the difference in the increased demand on the size of the meter. We have several such conditions that cannot be changed; therefore, this increased demand charge for the large meter goes on forever, or as long as the service is in existence.

In setting meters in factories, schools, or institutions they should always be by-passed and so gated that they can be easily tested in the line. If meters are so set they can easily be tested at least once a year, but if they have to be taken out and back to the shop for testing—you just don't do it, or do you? I know I didn't and I'll wager 99 per cent. of the water works men do not take out large meters and take them back to the shop each year or semi-annually for a check up. Personally I feel it advisable, and I strenuously recommend to put more into the setting of the meter than into expensive and cumbersome testing apparatus at the shop.

Perhaps I have not said a great deal as to how, in my opinion, one can determine for almost a certainty the proper size of meter to be selected. For instance,—a new factory building or institutional building is being constructed; the nature of its business will determine to a certain degree whether large or small quantities of water will be used. The length of line and pressure are also factors to be considered. In talking it over with the customer don't talk in terms of cubic feet. Cubic feet,—well he just can't comprehend what it means. Gallons,—yes, that is better especially in terms of 5 or 10 gallons, but not in terms of hundreds of gallons. I find that talking in terms of barrels is much better understood as the customer can visualize a barrel, and if you tell him that a barrel holds approximately 50 gallons he can visualize that much better. You ask him if he thinks he is going to use one full barrel each minute or 15 in a minute. You will then get from the customer a pretty good idea of

just how much he is going to use per minute,—whether he is going to use all large quantities or all small, or some of both.

With this information and with your experience in similar cases, you finally determine in your mind that he is going to use about 120 g.p.m. as a maximum. This is greater than the safe capacity of a 1½-in. meter and less than that of a 2-in. meter. I would set the 2-in. meter and after it is in service determine for a certainty if the right size has been selected by connecting on to the register of the meter a recording device that through its mechanism will record on a 24-hour chart (or a 6-hour chart) every gallon of water used, at whatever rate, and the number of times drawn. For a complete record the recorder should be run for one week. A record of consumption, demand periods, and rate of demand will be obtained. Then, and only then, in my opinion will you know if you have set the proper size of meter.

You may find that the night rate is nothing, the day rate 35 to 40 g.p.m., and so perhaps, in this case a 1-in. meter is ample in size. Take out the 2-in. meter and install the 1-in. meter. Your installation is less costly, and theoretically the consumption or registration should be larger and better with the smaller meter. However, this does not always hold true.

There has recently been placed on the market a recording machine that can be connected to any size or make of meter simply by removing the register cover and connecting it to the lowest index hand on the register. Registration goes on just the same, it is not disturbed in any manner and at the same time flow is being recorded on a 6- or 24-hour chart. Besides using this device for determining the proper size of meter, we use it for complaint work, such as large bills that cannot be accounted for. We find it an indispensable piece of equipment. It picks up conditions and reasons for large consumptions that the ordinary inspection does not discover.

WATER SUPPLY OF NORTHBOROUGH, MASS. AND ITS IMPROVEMENT.

BY ROBERT SPURR WESTON.*

[*Read December 17, 1936.*]

For more than fifty years the Town of Northborough has been supplied with water from Cold Harbor Brook in Shrewsbury. Recently, a water purification plant, chlorinator, new standpipe and water supply main have been added to the works.

Historical. The older works were proposed early in 1882 by a group led by D. B. Wesson and Cyrus Gale, Esq. On March 6 of that year a committee was appointed to procure surveys and estimates of cost. This resulted in a report by Percy M. Blake, C. E., whose recommendations the town adopted. The Enabling Act of the Massachusetts Legislature on April 29, 1882, followed, and on June 1 of the same year the town accepted the Act, appropriated $50 000, and appointed the committee to be water commissioners. Under Mr. Blake's direction, work was begun on August 2, 1882 and was finished the following year.

On February 17, 1883, the First Report of the Board of Water Commissioners was published. This was a complete account of the work and included regulations, rates, specifications, bids, contracts, cost data and Mr. Blake's final report. The latter gave the then cost of the work at $58 150.44. The value of the works before the recent improvements was $101 225. The present value is $204 307.

The first works consisted of a reservoir on Cold Harbor Brook and a distribution system, the latter including some cement-lined iron pipe.

On March 3, 1902, the town appointed a special committee of five to consider the question of additional water supply, and early in 1903 transmitted with its proposal two reports of James F. Bigelow, C. E., of Marlborough, recommending an additional reservoir storing 5 700 000 gal., the stripping of the lower reservoir and certain other improvements. One reason for the work of this committee was the location of an objectionable farm near the reservoir. This farm was later moved to the upper part of the catchment area.

The additional reservoir was not built, but in 1904 a by-pass of 10-in. vitrified pipe with head works was laid through the lower reservoir so that it might be drained and cleaned without interruption of the supply. This by-pass is no longer in service.

In 1929, there appeared the students' thesis of H. G. Butterworth and C. H. Carlson which contained various data regarding the supply. This

*Of Weston & Sampson, Consulting Engineers, Boston, Mass.

NORTHBOROUGH, MASS.
LOCATION PLAN OF WATER SUPPLY
0 MILES 1
SCALE
FIG. 1.

the writer has taken the liberty of using in part in preparation of this description. Mr. Carlson is now clerk of the Board of Water Commissioners.

Population. Butterworth and Carlson estimated for the town a water consumption of 90 g.p.d. per capita which is a liberal allowance. In 1902, Mr. Bigelow estimated about 34 g.p.d. per capita which even at that time was too low for a town of the class of Northborough. Ordinarily, the consumption of towns of about 2 000 population will range from 25 to 110 g.p.d. per inhabitant. This does not mean gallons per consumer because the rural towns have a lower consumption per inhabitant than the thickly settled industrial ones.

The capacity of the lower reservoir is 14.26 m.g. and the safe draft, therefrom is 12 m.g. The capacity of the upper reservoir, now removed, was 4 m.g., of which 2 m.g. could have been drawn.

Cleaning of the reservoirs, chemical treatment to control algae, a small subsiding basin at the inlet of the lower reservoir, and the ultimate purification of the water by filters were recommended by Butterworth and Carlson, who also recommended a larger spillway at the lower dam in the interest of safety.

The population of Northborough, as recorded in the past, and estimated for the future, is as follows:

Year	Population	Year	Population
1900	2 164	1925	1 968
1905	1 947	1930	1 946
1910	1 713	1935	2 060
1915	1 797	1940	2 125
1920	1 753	1945	2 200
		1950	2 275

Northborough is an agricultural, residential and manufacturing community. While agriculture is on the increase, this means that with the increasing use of machines and decreasing use of labor and draft animals on the farms the provision for the supply of water to them will not be greatly increased, excepting perhaps for intensive market gardening.

The manufacturing population has declined, while the residential population has increased, the resultant being a loss in population until 1910, followed by a slowly increasing population since then.

Because the supply is gathered from a catchment area which yields water of high color and on which the population is increasing rapidly, the supply has not been a satisfactory one. While other factors enter into the comparison, it is notable that in twenty years Northborough, with its unsatisfactory water supply, has increased in population but 14 per cent., while in the same period the populations of Westborough, Leicester, West Boylston, Holden and Shrewsbury, all of which have excellent supplies, have increased 26, 37, 67, 80 and 250 per cent., respectively.

Water Consumption. After the estimate of 90 g.p.d. per capita was made, the town installed a water meter in connection with an automatic device (Proportioneers, Inc.) for feeding chlorine in the form of H. T. H.* Meter readings showed much higher consumptions than the estimate of 90 g.p.d. per capita or the usual requirements of a town like Northborough. This unusually high consumption may have been due to unmetered connections or to leakage from the old cement-lined supply main, recently replaced.

Yield of Sources. The accompanying map (Fig. 1) shows the catchment area of Cold Harbor Brook above the reservoir and the locations of principle structures. The reservoir has a capacity of 14.26 m.g. and a safe draft of 12 m.g. Formerly there was a smaller reservoir (4 m.g.) located about one mile above the lower or main reservoir. In 1935, the Board of Water Commissioners improved the brook above the lower reservoir. This improvement included the rip-rapping of the banks, the drainage of swamps and taking down the dam of the upper reservoir, thus reducing the available storage from 12 to 10 m.g.

It is estimated that to obtain a yield of 180 000 g.p.d., which must be provided before many years unless consumption be decreased, the reservoir must be drawn upon to the limit of desirability.

Additional Storage. It would be well if storage enough could be provided above the lower reservoir so that it could be kept more nearly full at all times than is otherwise possible. The reservoir recommended by Mr. Bigelow in the Report of the Special Committee in 1903 would store 5.7 m.g. and would increase the safe yield by about 43 000 g.p.d., assuming, however, the drawing down of the lower reservoir by 8 ft., which would be unwise. An additional storage reservoir will probably be required within twenty years, although metering would extend the time.

Quality of Water. As shown by the records of the State Department of Public Health, the water supply of the town is disagreeable in appearance, and unless disinfected, unsafe for drinking. Its bad appearance is due to its high color, and its unpleasant odor and taste. The color is due to dissolved vegetable matter (swamp tea). It is highest when rains fall upon swampy land after a long, dry spell.

Samples collected by Weston & Sampson in 1934 from various points within the water-shed showed colors varying from 10 to 160 p.p.m. While storage reduces this color somewhat, the average of that of the water at the dam is high, the results of analyses by the Department of Public Health showing an average of 74 in 1935 and those by Weston & Sampson an average of 54 since the plant was placed in operation in September 1936. The main source of the color is the swamp along the brook crossing the

*High Test Hypochlorite.

Shrewsbury-Boylston line, one examination showing that 80 per cent. of the color came from this source.

Dissolved vegetable matter and growths of microscopic organisms produced disagreeable tastes in the water. Repeated attempts to eliminate these odors by cleaning the brook and reservoirs were helpful but not preventive. The necessary chlorination of the water for hygienic reasons accentuated the odors.

Change in Quality. While, on the whole, the appearance of the raw water has been better during the last few years than during the first years of the life of the works, pollution has been increasing on the catchment area, particularly west of Rawson Hill in Shrewsbury and along the Shrewsbury-Boylston road where, because of the attractive location, a considerable number of new houses has been built in the last decade. The large Masonic Home is also within the watershed although its sewer is carried beyond it, into the drainage area of Lake Quinsigamond.

Chlorination. Following the advice of the State Department of Public Health, a portable chlorinator was installed on March 29, 1934. During 1935, permanent devices were installed along the supply main, at a point below the junction of Cold Harbor Brook and its tributary leading from Rocky Pond, this location being chosen so that in an emergency chlorinated water could be drawn from the tributary. The chlorinator was made by Proportioneers, Inc. of Providence, R. I. and consisted of a Chlor-O-Feeder, pumping solution into the main from a supply tank and controlled by the water meter on the supply main.

Other Sources of Supply. Early in 1934, the town appointed a Water Investigation Committee of which Mr. Frank W. Eames was Chairman. This committee engaged the writer's firm to make a study of conditions and to recommend improvements. A report was submitted on May 21, 1934, in which the water purification plant and standpipe described in this paper were recommended. These recommendations were made after other sources of supply, such as ground water and Metropolitan works, had been considered; also the improvements that might be gained by cleaning the reservoirs and draining and clearing the swamps.

Slow versus Rapid Filters. While for such a highly colored water, chemical treatment and filtration through rapid filters would seem to be indicated, the location of the plant on a bad road in the hills, the prohibitive cost of constant attendance and the general reliability of the slow filter with coagulation led to the choice of this type. In principle it is similar to plants at Springfield and Greenfield, although of course much smaller.

The general arrangement of the plant which is located at the westerly end of the dam is shown on the accompanying sketch (Fig. 2). Water

is taken from an intake in the reservoir, through an 8-in. suction main, to a motor-driven centrifugal pump discharging through a chemical-mixing tank into the coagulating basin. This pump has a capacity of 360 g.p.m. and is located in the basement of the filter house.

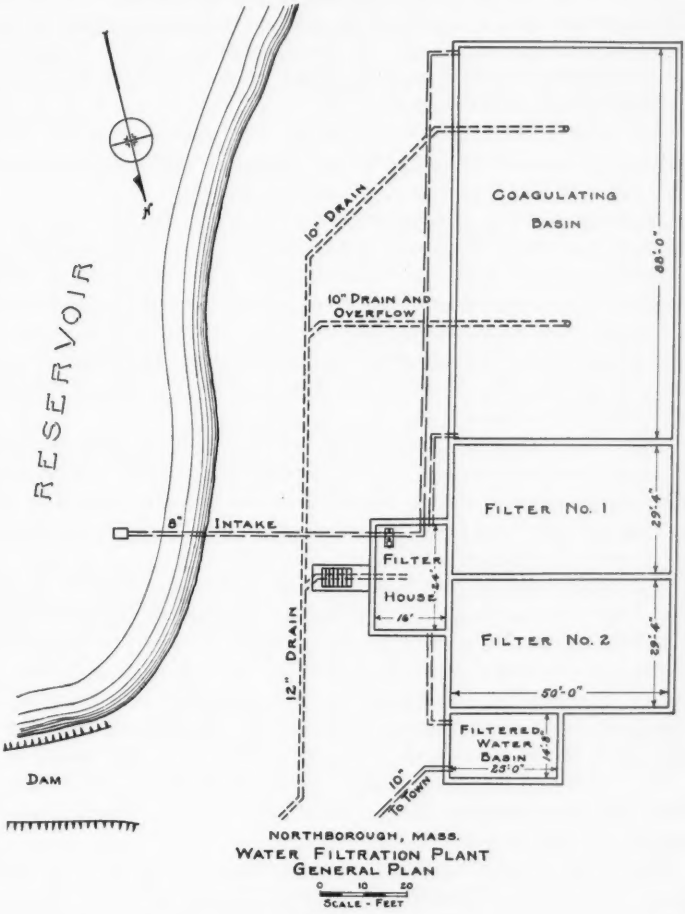

FIG. 2.

From the pump, the water is discharged into a mixing tank into which an Omega dry feeder* discharges sulfate of alumina as required. The discharge from the pump enters the mixing tank at the bottom, then crosses the tank and flows out from its bottom through an 8-in. pipe to the coagulating basin. The chemical is applied into the outflowing stream.

*Since this paper was presented, a second feeder has been installed for adding alkali or sodium aluminate to the raw water at times of low natural alkalinity.

Water enters one end of the coagulating basin through a mixing trough designed to distribute the flow, and exits through an outlet trough, at the opposite end, to the two filters. The coagulating basin is 88 ft. by 50 ft. in area and when filled to its flow line has a capacity of 395 000 gals.; when filled to its overflow it has a capacity of 403 000 gal. With filters operating at 5 m.g.d. per acre, it provides a coagulating period of more than 24 hr. after due allowances for short circuiting. This long period is designed to minimize the effects of imperfect treatment, which would result in too rapid clogging of the filters.

Each filter is 50 ft. by 29 ft. 4 in. in plan, equivalent to an area of 0.033 acre, or a total area of 0.066 acre which is equivalent to a rated capacity of 0.33 m.g.d. at a rate of 5 m.g.d. per acre.

As shown in the accompanying sketch (Fig. 2), the filters are of the usual slow sand type. They are built of reinforced concrete with flat-slab roofs supported on beams, walls and columns. The bottoms of the filters are in the form of inverted groined arches with the drainage channels therein covered with split, vitrified-tile pipe. Above is a depth of 12 in. of graded gravel supporting a layer of sand 3 ft. in thickness. The depth of water above the sand is 4 ft. The filter sand has an effective size of 0.34 mm. and a uniformity coefficient of 3.87.

The effluent from the coagulating basin enters each filter through a float valve located in an inlet well at the corner of the filter. This well is in the form of a quarter circle. Its wall is of concrete surmounted by three rows of brick, which can be removed successively as the surface of the sand is lowered by scraping.

The piping, valves and connections are in the basement of the filter house. The piping includes various drains and connections, including one for backfilling the filters after scraping. Each filter is provided with a Builders Iron Foundry controller with the usual indicating rate-of-flow and loss-of-head gages, these latter being located above on the main floor of the filter house. Gage glasses are provided to show at all times the elevations of water in the coagulating basin, filters and filtered-water basin.

On its way to the filtered-water basin, the water receives a dose of sodium carbonate from an Omega feeder located in the filter house. This chemical is added to minimize corrosion.

The filtered water basin is of concrete 25 ft. by 14 ft. 8 in. in plan and has a capacity of 28 600 gal. It is designed to provide storage for filter operation only, storage for domestic supply and fire hydrant service being provided in the standpipe. From the filtered water basin the water passes through a 10-in. main connecting with the original 12-in. main from the reservoir to the town.

Standpipe. The standpipe is located near the top of Mt. Assabet near the center of the town and has a capacity of 500 000 gals. It is of a common type with flat bottom and conical steel roof. It has an inside

diameter of 45 ft. and an inside height of 42 ft. It is connected with the 10-in. main on Main Street through a 12-in cast-iron pipe, 854 ft. long, laid through the Park and streets to Main Street.

A double-acting, automatic, cushioned altitude valve was provided to control the water level in the standpipe. This valve is designed to close when the water level in the standpipe reaches elevation 439.0 ft., which gives a static standpipe pressure on the center of the valve of 46.84 ft., and to open wide when the pressure on the distribution side of the valve drops about 1 ft., below the standpipe pressure.

Automatic Control. The filter being designed for automatic operation and against the possibility of its being unattended for 48 hours or longer, the various float valves and switches required were selected with care. The elevation of the water in the coagulating basin is controlled by a float switch which in turn controls the operation of the pump and the coagulant feeder. The elevation of the water in the filters is controlled by float valves on the influent pipes. As water rises in the filtered water basin, it rises in the filters. When the latter become filled, the inlet valves close, thus raising the elevation of the water in the coagulating basin, closing the float switch and stopping the pump and chemical feeder. These latter start under reverse conditions. At present the sodium carbonate feeder is set for a constant rate.

General Features. The filter house is of a dark red brick with cast stone trimmings; the control room is lined with a gray faced brick; the roof is of slate. An oil heater and arrangements for lighting are provided, also storage for a small quantity of chemicals, main storage being provided elsewhere.

Operation. The plant was placed in service on September 1, 1936, and has been in continuous operation since. During September and October, considerable difficulty was experienced because of the rapid clogging of the filters. This condition, however, was not peculiar to Northborough but obtained at several water purification plants in New England. It was due to the character of the vegetable matter in the water. Considerable growths of algae have existed in the reservoir also. Both these conditions shortened the filter runs for a considerable period.

The results of operation have been excellent. On November 23, 1936, with the filters operating at an average rate of 121 000 g.p.d. each, samples were collected for analysis from the reservoir, the filter effluents (mixed) and the tap at the chlorinator, respectively, with the results shown in the accompanying table.

The results show complete removal of the distinct musty-vegetable odor and the turbidity from the reservoir water and the reduction of its color from 70 to 5 p.p.m., equivalent to a reduction of 93 per cent. The

RESULTS OF ANALYSES.

Determination.	Parts per Million.*		
	Reservoir.	Filter Effluents.	Tap at Chlorinator.†
Odor — Cold — 20° C.	3 M. — Veg.	None	None
Odor — Hot — 90° C.	3 M. — Veg.	None	None
Turbidity — Silica Standard	3	0	0
Color — Platinum Standard	70	5	5
Oxygen Consumed	10.1	3.0	2.9
Nitrogen as Free Ammonia	.014	.006	.012
Nitrogen as Albuminoid Ammonia — Total	.246	.090	.082
Nitrogen as Nitrites	.006	.001	.001
Nitrogen as Nitrates	.020	Absent	Absent
Chlorides — Cl	2.0	2.0	2.0
Hardness, by soap method	20.0	20.0	20.0
Hydrogen Ion Concentration — pH	6.8	5.4	7.8
Iron — Fe	0.28	0.05	0.07
Residue on Evaporation — Total	76	56	79
Mineral Residue — Total	38	39	58
Carbon Dioxide (free)	3.5	8.0	0.0
Carbonate Alkalinity	.....	.....	2.0
Bicarbonate Alkalinity	.....	.....	21.0
Bacteria per c.c. at 20°C.	36	0	0
Bacteria per c.c. at 37.5°C.	22	0	1
Coli-Aerogenes Group:			
Confirmed Test	+ in 10 c.c.	— in 50 c.c.	— in 50 c.c.
Presumptive Test in 5, 10-c.c. portions	4/5 Pos.	5/5 Neg.	5/5 Neg.
Alkalinity — as CaC 0_3	8.0	2.0	23.0
Presumptive Test in 1, 1.0-c.c. portion	1/1 Neg.	1/1 Neg.	1/1 Neg.
Presumptive Test in 1, 0.1-c.c. portion	1/1 Neg.	1/1 Neg.	1/1 Neg.

*Unless otherwise stated.
†Chlorinator not running.

addition of sodium carbonate resulted in raising the alkalinity to 23 p.p.m. and the pH value to 7.8. Bacteriological examination showed the reservoir water to be slightly polluted and the filtered water to be sterile. The condition of the filtered water permitted the discontinuance of chlorination.

During 1937 the appearance of the effluent has been even better than shown in the preceding, zero colors being obtained at times.

Occurrence of Mosquito Larvae. In September 1936, numerous aquatic larvae, *Chaoborus* (*Corethra*), were found on top of the filters where they had concentrated just prior to scraping. These rare organisms are phantom larvae of a carnivorous mosquito. This mosquito is not commonly known because it does not suck blood. According to the authorities, it is the only insect found with the plankton in open water. Figure 3 shows this insect. Its notable characteristics are two pairs of balancing sacs, one near each end of its body.

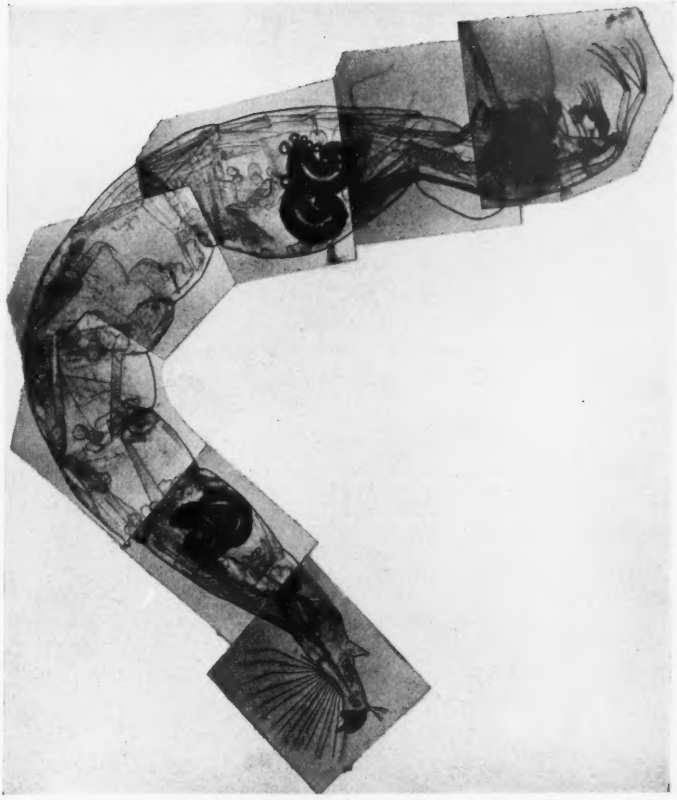

FIG. 3.— LAVA OF *Chaoborus*.
Photographed by H. C. Lithgoe of the Massachusetts Department of Public Health.

Construction Cost. The approximate cost of the new works, with engineering, was as follows:

Water purification plant	$45 166
Chlorinator	1 000
Supply main, 12-inch	59 464
Standpipe and foundations	13 357
Standpipe connection and Altitude valve	3 881

The total estimated cost of the works, including engineering, contingencies and land takings, was $126,461.

The work was performed under the Public Works Administration, Docket No. Mass. 1041-R. The water purification plant and standpipe foundation were constructed by the Morgan Construction Company of

Boston; the standpipe was built by the Bethlehem Steel Company of Boston; filter equipment was furnished by the Roberts Filter Manufacturing Co. of Darby, Penna.; and the chlorinator was furnished by Proportioneers, Inc. of Providence, R. I. The supply main was installed by A. Piatelli & C. Di Martino.

Whitman & Howard of Boston were engineers for the supply main, and Weston & Sampson of Boston for the water purification plant, standpipe and connections. Mr. Ralph C. Henry of Boston was consulting architect for the filter house.

Mr. John W. Allen, Clerk of the Board of Selectmen, represented the Town of Northborough, and Mr. G. F. Neuberger was the P. W. A. Resident Engineer Inspector on the work.

Mr. A. C. Libby was resident engineer for the water purification plant and standpipe; Mr. Paul S. Barry for the supply main.

PROGRESS REPORT OF COMMITTEE TO STUDY THE ADVISABILITY OF WATER DEPARTMENTS REGULATING WATER PIPING AND FIXTURES WITHIN BUILDINGS.

[Read September 23, 1936.]

The Committee appointed to study "The Advisability of Water Departments Regulating Water Piping and Fixtures within Buildings" is agreed that the term "service piping" might better be used which includes all material from the water main to and including the fixture supplied and that the scope of this study should be assumed to include that part of the service between the street main and point of entry into the building.

The following matters have been discussed and are listed to bring to attention pertinent existing features and common faults which should in general be remedied in existing installations and which by proper coördination of interested parties may be prevented in new work:—

1. Size, type, location and material used for service.
2. Types of fixtures — with reference to possible contamination of the water supply or pollution caused by back syphonage — including sinks, drinking fountains, flush toilets, urinals, bath tubs, hospital sterilizing equipment, etc.
3. Special equipment such as used for circulating and purification of water supplied to swimming pools, sewage ejectors, sewage pumps, sump drains, chlorinator equipment for dosing of sewage, hydraulic gasoline dispensing equipment, commercial laundry tubs, chemical vats, air conditioning equipment, etc.
4. Storage tanks used for pressure boosting or equalizing in high buildings.
5. Proper construction of domestic hot water boilers and adequate protection by pressure-relief and thermostatic-control valves.
6. Protective electrical grounding on water pipes with reference to location of grounds and protection against shock to workmen on meter installations.

The Committee invites a full and free discussion of these matters which pertain not only to the protection of individual installations but also to the protection of the purity of water in the street mains. It is the duty of water works men to initiate and support measures to see that proper protection is afforded against such possible pollution and that in the future new installations are provided with proper protection. It is appreciated, however, that while in certain municipalities it would be comparatively easy to adopt regulations for inspection and enforcement, such as have been placed in effect in the City of New Bedford, other municipalities might have considerable difficulty in establishing a similar procedure on account of varying state laws and municipal ordinances.

It is therefore suggested that the best method of attack would be for each water works management to study the set-up in its individual com-

munity or group to determine which of the several administrative agencies or departments could take over the functions of establishing regulations or codes and conduct the necessary approval of plans and inspection of new work.

This preliminary report is intended to promote discussion on these matters. The Committee would especially welcome discussion by representatives of private water companies as to their attitude on this question. We would suggest that a winter meeting be devoted to this subject after which it is thought that it will be possible to draft a more comprehensive outline of suggested regulations which might serve as a guide to all concerned.

Signed, STEPHEN H. TAYLOR, *Chairman*
HARRY U. FULLER
DAVID A. HEFFERNAN
WARREN J. SCOTT
ARTHUR V. HARRINGTON

WHAT INTEREST HAS A WATER PURVEYOR IN INTERIOR PLUMBING SYSTEMS IN RELATION TO CROSS-CONNECTIONS.

BY M. W. COWLES.*

[*Read November 19, 1936.*]

Adequate water service throughout a building is important to the occupants of the building and to the water purveyor as well. The scope and purpose of this meeting under the auspices of your Committee on Control of Interior Piping Systems, is very broad. The writer wishes to direct your attention to some of the health aspects which are a part of the problem you are considering. The effort made to provide safe drinking water in the mains is ineffective if water delivered from taps within a building is not equally safe at all times. It is doubtful if the majority of water purveyors or health officials realize the numerous possibilities for the contamination of tap water that exist within buildings. Such a condition may occur as a result of a chain of circumstances, which produce pressure differences that, in turn, make possible the flow of polluted water into the water distribution pipes either by pressure or by siphonage. Can the water superintendent assist in preventing such occurrences?

The number of types and the variety of cross-connection that exist in our buildings are legion. No attempt will be made in this paper to enumerate them. The danger from cross-connections first became apparent when the use of auxiliary water supplies for fire protection became necessary in large factory buildings to supplement fire protection afforded by public water supplies. There are direct connections between public and private water supplies. There are connections between water pipes and drain pipes or sewers, direct and indirect. Still another group is that of plumbing fixtures and other water-connected devices where cross-connections of various types are built into the fixtures or devices.

In 1928, the New Jersey State Department of Health passed a regulation, with the force and effect of law, requiring cross-connections then in existence to be provided with specified protective devices on the basis of a permit issued on a year by year basis. The regulation was intended primarily to apply to connections between public water supplies and other auxiliary sources of water supply. The Hackensack Water Company, supplying 52 towns in northeastern New Jersey, became active in investigating commercial and industrial premises where cross-connections were thought to exist by reason of known auxiliary water supplies for fire or industrial uses. About 250 cross-connections were found and eliminated prior to 1930. Under the procedure provided by the state

*Health Officer, Hackensack Water Company, New Milford, N. J.

code, applications were filed and permits granted for the continuation of some 17 connections, almost all of which were on fire lines with auxiliary sources of supply. The New Jersey code required that each such connection must be provided with two all-bronze check-valves of the Factory Mutual type. Tests for tightness, required four times a year, are used as a basis for the application to the State Health Department for renewal of the permit for another year. Our quarterly tests have generally shown tight valves. There have been a few decided exceptions but in no case, have both of the valves been found leaking at the same time. Due to changes of use, or reconstruction of portions of industrial plants, many of these existing permitted connections can be eliminated in time. When applications for new services to industrial plants are filed with the company, conferences are held with the applicant, the problems discussed and arrangements made to construct the new piping system so that no cross-connections will exist. About a year ago, this procedure was extended to apply to all services where an auxiliary water supply was thought to be in use.

Reinspections of factories, where cross-connections were found and eliminated in 1928 and 1929, indicated that additional cross-connections had since been made. A change in personnel in a particular industry, may have been responsible in part, as well as changes in water-piping systems incidental to growth. These experiences pointed clearly to the fact that, even if cross-connections had been eliminated, there was no assurance that new ones would not be created. Accordingly, in December 1934, a full-time inspector was appointed for this work, functioning under the direction of the writer. We have endeavored first, to secure a representative picture of the cross-connection problem on our system including some 73 000 services supplying residences, commercial buildings, institutions, and industries of various types. As a result of this study, a definite program of activities is being followed. The work is divided into three parts: (1) inspection to locate existing cross-connections and secure their elimination; (2) prevention of cross-connections in new buildings and on new services and (3) studies in regard to plumbing fixtures and all other types of water-supplied equipment which are so designed as to constitute cross-connections. Written reports are prepared for each inspection accompanied by detailed sketches showing the layout of the interior piping system from the meter to all water outlets. As eliminations are secured, or as changes in the piping system are made, the sketches are corrected. Every commercial and industrial building inspected, is reinspected at regular intervals, from once in three months to perhaps once a year, depending upon the use of the building or premises and other factors. These reinspections are controlled through a card index.

Industries are generally large consumers of water and, therefore, constitute the best customers of the water purveyor; they deserve specialized service. In those industries where permitted cross-connections with

the double check valves now exist, the management and the operating staff are, of necessity, familiar with the problem, and little preliminary educational work has to be done. In other industries, it is necessary to first approach the management and to discuss with them the dangers to health which may exist by reason of unknown cross-connections in their plant. A complete inspection of the plant is then suggested, and from their maps and from field observations a map or sketch showing all the water piping system or systems is prepared. All cross-connections are located and shown on the map as well as listed in tabular form. As soon as this is completed, a conference with the management and some of the operating personnel is held to talk over the problems involved. At such a conference, reference is made to actual cases of pollution by cross-connections which have been found on other customers' premises or have occurred elsewhere in the country. The management readily realizes that illness, due to contaminated water, is a serious matter from several angles and may involve them with workmen's compensation laws. They soon become only too anxious to eliminate the objectionable conditions and coöperate to the fullest extent.

In connection with large factories where several different water supplies are used, the methods suggested for actual elimination must be based on study of their water requirements. Auxiliary sources of supply of very large capacity, may be vital for certain industries. Maximum flexibility in the use of water from different sources is essential, yet no cross-connections should be created. Complex operating problems can sometimes be readily simplified by minor changes in one or more of the water systems. A water purveyor is in the business of selling water, but service is also equally important. The specialized service rendered by the water purveyor is greatly appreciated by the industries. The water purveyor must realize the inter-relation of all these factors and must not be averse to the customer's idea of having the public water supply available only as a standby, if free from cross-connections. Friendly relations with customers is most important and, in the long run, works out to the mutual advantage of both purveyor and customer.

The regulation of the New Jersey State Department of Health, previously referred to, has been a vital back-log behind our activities. Much less could have been accomplished in the absence of such a regulation. The eliminations made have been secured without legal action which is possible under this regulation. Dependence has been placed on persuasion, and appeal to reason. Appreciation of the hazards produced by cross-connections readily convinces the property owner or factory manager that the request for the elimination is entirely reasonable and of more benefit to him and his organization than to the water purveyor. The development of this cordial mental attitude goes a long way toward preventing establishment of other cross-connections in the future. These few words describe the general situation but it has been occasionally

necessary to secure the assistance of local boards of health and even the State Health Department in backing up our request for elimination.

The campaign against cross-connections in internal piping systems of all classes of buildings is largely one of education. All of the personnel of a water purveyor must become somewhat familiar with cross-connections, as the staff, in their contacts with customers, may frequently pick up information of great value, even though its duties may not be even indirectly related to inspection. As cross-connections are found in any of the 52 municipalities that we supply, contact is made with the local health representatives at a proper time for the purpose of acquainting them with the particular situation. Subsequent contacts make possible the creation of real interest in the problems of cross-connections with the realization that they, as health officials, should be as much or more interested or concerned in the cross-connection problem than the water purveyor. This in turn, leads up to a request to the water purveyor for suggestions for suitable amendments to the local plumbing codes or sanitary codes, a service which the water purveyor should be best qualified to render. The essential element of such codes is the requirement that the water distribution system in all buildings be considered a part of the plumbing and drainage system. Cross-connections can be covered in a few short paragraphs of a very general nature. With the water distribution pipes a part of the plumbing and drainage system, it would seem that the problem would be solved but it is not that simple. You would be very much surprised to find how many representatives of organizations, installing different types of water-supplied equipment, are allowed to tamper with the water pipes through improper wording of certain code paragraphs. You would be equally surprised at the possibilities of back-siphonage in a large percentage of the plumbing fixtures now on the market. The number of other types of water-supplied equipment of various kinds, other than plumbing fixtures, is rapidly increasing. Much study must be given to many phases of this problem to effect a solution.

Architects, particularly those designing hospitals, are becoming very much interested in cross-connection prevention. At present, their efforts must be confined to the installation of vacuum breakers and other devices on standard plumbing equipment. A mechanical air-break, which may get out of order, is unfortunately not a substitute for proper design of fixtures. Some plumbing contractors have gone so far as to make their own tests to determine whether or not one type of fixture can be more readily siphoned than another. At least three of our larger cities have testing laboratories for such devices. Interest is increasing. The work of several of the state health departments has been of great assistance. The National Association of Master Plumbers has been and will continue to be helpful. The American Public Health Association is beginning to show considerable interest. All of this activity holds out a ray of hope to water purveyors. Some much more effective coördination of activities

along these lines must be established. Otherwise numerous administrative difficulties may arise, nullifying the progress thus far made.

The remarks previously made on the necessity of continuing inspection, merit further discussion. Plumbing is installed, inspected, and tested before use, but that gives the health authorities or water purveyors only a false sense of security against dangerous conditions that may arise in the future. Inspection on a regular basis is, to my mind, an essential or perhaps better, a vital part of the cross-connection prevention program. This thought brings up the very definite question of who is to do this inspection work. The duty of plumbing inspection usually comes under the jurisdiction of the local health officials, and likewise reinspection of plumbing, all of which is for the protection of public health. The water purveyor should be ready and willing to coöperate in every possible way in solving the numerous water-service problems that arise. Dual water supplies in factories and the elimination of cross-connections are more or less engineering problems. The water purveyor can and should help in working out these situations, not only to eliminate the cross-connection, but to enable it to provide the best possible service to the customer. Duplication of activity between different public agencies is highly undesirable. By coöperation and carefully planned coördination, much can be accomplished toward the desired end.

Records of customer complaints are of inestimable value in the cross-connection problem as well as in all other activities of the water purveyor. Some cases of alleged illness reported may be traced directly to cross-connections and without such reports might not be detected. Our Central Service Bureau established in 1929, provides a clearing house for all types of complaints and the investigations that have been made in answer to them. The information on file in that office, is of the greatest possible value in providing adequate service to the customer. Even in a small public supply, written reports of complaints are preferable to memory. Such records should include references to poor pressure or flow, overheating of boilers, possible damage to meters, and many other complaints, too numerous to mention, with many of which your Committee is concerned. The necessity for and the value of such detailed written records of complaints cannot be emphasized too strongly.

Service to the customer, one of the broad policies of our Company, established by its late President, Nicholas S. Hill, Jr., has been of the greatest assistance in establishing confidence on the part of our consumers. Confidence almost always produces coöperation. Without this attitude on the part of the consumers, much of the work on cross-connections would have been impossible. Maintenance of proper public relations is equally possible whether the water purveyor is publicly or privately owned. The success of any cross-connection problem is based on customer relations and on the general attitude of the individuals in charge of policy of the water purveyor and on the attitude of its operating personnel rather than on mere ownership.

PLUMBING FIXTURES SUBJECT TO BACK-SIPHONAGE.

BY LESLIE K. SHERMAN.

[*Read November 19, 1936.*]

The possibility of a vacuum in water-supply piping is quite generally realized, and although fortunately not of frequent occurrence, many water works men have undoubtedly at some time noted such a condition.

Excessive friction loss in water-supply piping in buildings, caused by inadequate pipe size or corrosion, will result in lowered pressures or vacuums in upper floors when water consumption is heavy on lower floors. The probability of vacuums from this cause is increased with the age of piping and building height. A break or shut off of the street main may cause a serious vacuum. That this occasionally happens is evidenced by collapsed hot-water tanks and cooling coils. When a vacuum does occur, a suction is created at faucets or points of normal discharge. If at such a time the water line is open, air will usually enter the line to relieve the vacuum. However, if the point of discharge is submerged, the liquid covering the water outlet will be forced into the water pipe by atmospheric pressure.

Any plumbing fixtures that permit the water inlet to be submerged under normal or unusual conditions are potentially dangerous, and effort should be made to eliminate any possible health hazard from such cross-connections by requiring the use of only properly designed fixtures or protective devices.

In the remainder of this paper an attempt will be made to point out fixtures in which potentially dangerous cross-connections may occur.

I. TOILET BOWLS AND URINALS.

A. Toilet bowls and urinals using flushometer valves not properly protected permit cross connections:

1. Normally, if of the side-inlet type.

2. Abnormally, if a stoppage occurs in the top-inlet type. The discussion of "Cross Connections in Plumbing and Water Supply Systems" published by the Wisconsin State Board of Health, recommends for these fixtures:

"1. A flush valve so designed that the main valve does not lift off its seat when a vacuum occurs in the supply line;

2. a vacuum breaker, with or without check, placed between flush valve and bowl, or bowl with vacuum breaker incorporated;

3. flush valve to be placed about 6 inches above overflow rim of the bowl;

4. vacuum breaker must always be above overflow level of the toilet bowl; and

5. a check valve may be combined with the stop as an added protective feature."

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B. Seat-Action Closet.

This type of closet is subject to conditions similar to bowls with flushometer valves and protection may be obtained in the same manner.

C. Tank Closets.

This type of fixture commonly permits a less objectionable cross-connection. Tank closets may be divided into two general types:

1. Tank above bowl type permitting cross-connection with the tank only, which, however, is undesirable as considerable débris may accumulate in the tank.

2. Tank integral with bowl type, in which water in a clogged bowl may mingle with water in the tank; if a cross-connection exists in the tank, the contaminated water from the bowl may reach the water piping.
Protection against tank water entering the water line may be provided by the use of a vacuum breaker on the discharge pipe or ending the discharge pipe above the tank overflow. In the latter case a muffler of some sort will be required in many locations.

II. Fixtures With Under-rim Inlets.

A. Bathtubs.

The worst type of fixture has the inlet below the overflow. Any below-rim inlet is a potential hazard, as the overflow may be inadequate, or rendered ineffective by a clogged trap. There is no apparent advantage to below-rim inlets in tubs.

B Lavatory Basins.

Inadequate overflow or a plugged drain will cause the inlet to be submerged. As in the case of bathtubs, no advantage is apparent with under-rim inlets.

C. Sinks.

Under-rim inlets are rare with these fixtures except in soda fountains or when a piece of hose is attached to the water inlet.

D. Laundry Trays.

Laundry trays with below-rim inlets are common, and frequently no overflows are provided. Even if an overflow is provided, it may be covered by clothes in the tubs.

E. Drinking Fountains.

The water inlet should be above the fixture rim to prevent possible back-siphonage with a clogged or inadequate drain. Attention is also directed to the danger of spread of disease among users of drinking fountains made possible by improperly designed fixtures. This is entirely aside from pollution of the water piping. Reference is here made to "Recommendations for Design of Sanitary Drinking Fountains" by the Joint Committee of the American Public Health Association and the Conference of State Sanitary Engineers, October, 1930.

III. Special Fixtures.

There are a great many special fixtures and piping arrangements that may cause cross-connections. Among them are:

A. Ejectors for sewage and cellar drains.

B. Refrigerator drains with flushing connections.

C. Condensers and coolers with direct water and sewer connections, also with regular and emergency connections with a secondary water supply.

D. Restaurant dishwashers.

E. Dairy bottle-washers.
F. Laundry equipment.
G. Pump-priming connections.
H. Pump water-seal connections.
I. Hospital equipment, including
    1. Sterilizers.
    2. Slop sinks with flushometer valves.
    3. Bidets.
J. Laboratory equipment, especially:
    1. Aspirators.
    2. Water baths.
    3. Stills.
K. Swimming pools.
L. Chlorinators.
M. Dental lavatories.
N. Manicure tables.
O. Vats and tanks in industrial plants.

It is possible to eliminate cross-connections in fixtures such as those mentioned by the use of properly designed above-rim fixtures, or the installation of adequate air breaks. Several means of providing air breaks are available, such as:

1. Venturi air breaks;
2. siphon-preventer check-valves;
3. funnel air-breaks; and
4. tanks with above-rim water inlets.

There appears to be no advantage to most under-rim inlet plumbing fixtures, and although a great many are offered for sale, it is of interest that at least one large manufacturer has eliminated this objectionable feature from basins and tubs displayed and will supply under-rim inlet fixtures only on special order. The existence of regulations against unsafe fixtures will arouse the interest of plumbers and the public, and all manufacturers will undoubtedly be prompt to supply only fixtures that make back-siphonage impossible.

Any investigator on the alert for dangerous back-siphonage plumbing connections will soon note the existence of unsafe conditions in the course of his every day experience. A few instances are noted here as examples.

In an investigation of the cause of poor bacteriological examination reports on a Connecticut water supply system, the possibility of cross-connections was looked into. It was found that on the third-floor bathroom of a rooming house there was an old fashioned bath tub with a bottom inlet. The vertical piping in the building had become corroded, so that when a

flow of water was started at the kitchen sink on the second floor, no water could be drawn on the third floor. An experiment was tried whereby the bathtub on the third floor was filled and the faucet left on; at the same time the faucet at the second floor sink was turned on. The result was that water siphoned out of the bath tub into the house piping.

Joel Connolly, Sanitary Engineer of the Chicago Department of Health, has reported some interesting personal experiences. Connolly states that on the fifth floor of a hotel in a Southern city,

"— I was drawing a glass of water. Just as the glass was half full, the water ceased to run and air was sucked into the open faucet. After a few moments the water pressure was restored and the glass was filled. Not caring to drink it, because of the likelihood that suction of polluting material into the water had occurred from other fixtures in the hotel, I set the full glass on the shelf. Shortly afterward, hearing the gurgling sound from the water closet indicating that the flush valve had been sucked open, I held the glass of water under the cold water faucet and opened it. The water in the glass promptly was siphoned into the supply pipes.

"On the sixth floor, periods of vacuum in the water pipes were said to be quite frequent. This occurrence was the more startling because a convention of water works men was in progress in the hotel, at the time, and it was said, an inspection made before the meeting had indicated that the plumbing system was safe."

The fact that vacuums in building-piping occur frequently is borne out by the writer's experience while living in a third-floor apartment. In this building, three stories high, a separate riser was provided for the flushometer valves. However, this riser was obviously inadequate, because a temporary vacuum would unseat the flushometer valve whenever a toilet was flushed on the second floor.

Undoubtedly, stories of similar experiences could be continued indefinitely if a real attempt were made to assemble such data, but suffice it to say that the dangers of plumbing cross-connections are intimately with us, and coöperative action should eventually bring about their elimination.

In many cases it is difficult to remedy existing plumbing hazards through laws or regulations, and education of plumbers, manufacturers, and the public is probably the most practical method of bringing about improvement; but even if replacement of fixtures frequently cannot be secured, it may be possible to promote installation of siphon breakers and air vents to eliminate the worst hazards. It should be feasible to prevent the installation of potentially dangerous types of new fixtures and new plumbing by the enactment and enforcement of suitable regulations.

SPECIAL EQUIPMENT CROSS-CONNECTIONS.

BY ARTHUR V. HARRINGTON.*

[*Read November 19, 1936.*]

The subject dealing with special equipment is largely a repetition of the topic covered by Mr. Sherman in the preceding paper in this Journal in that in almost every instance the possibility of pollution or contamination is as a result of siphonage.

There are any number of cross-connections to equipment which appear to be harmless, but I am certain that no one would appreciate having to use a supply of tap water which had previously been used in a swimming pool, even though it has been thoroughly chlorinated, previous to which it was passed through an uncertain filtration process where alum and other coagulants were used to provide a clear, sparkling water when placed in a beautiful tile pool. There is every chance that this might happen, as frequently the filling or replenishing supply connection for this equipment is made directly into the pool or to the discharge line of the circulating pumps where it is possible, under certain conditions, to build up a head equal to, or in excess of, that in the street mains.

Generally speaking, we think of chlorinating apparatus in connection with the purification of water supplied to our water systems, but there is an application of chlorinator equipment which very materially introduces a hazardous condition to any system of water pipes. I refer to the case where the chlorinator is supplied with water to make a solution feed for the final treatment of a sewage effluent. This arrangement provides a direct physical cross-connection between the water supply piping and the sewage-effluent line where, under a condition of negative pressure existing in the water supply line, it is possible to lift the sewage into the distribution line. Should this condition continue for any length of time, the sewage might reach the street main.

A similar hazard exists in those installations where sewage ejectors are used to raise sewage from a pit or pipe line at one level to a discharge point at a higher level. Probably everyone is acquainted with the operation of an ejector, but it might be well to point out that there is no separation between the chamber that contains the jet orifice and the secondary chamber. When in operation, the water passes through it at high velocity into the secondary chamber, producing a vacuum which in turn lifts the sewage, the discharge from the ejector being a mixture of sewage and clean water. The fact that there is no separation between these chambers and that the device is connected into a tight system of piping provides the set-up that would permit back siphonage under the proper conditions.

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The same situation is true when an ejector is used to lift drainage water or wastes from a sump. The possibilities of pollution in these cases are the same, the only difference being that the material handled may be somewhat less harmful than sewage.

In many manufacturing plants where chemicals are used in processing, frequently large vats, tubs, hogsheads or other containers are used to store the chemicals in solution, and very often a pipe line or hose is dropped into the vat from which water is supplied without spatter for purposes of mixing. In these circumstances, a perfect condition is set up whereby the contents of the vat, tub, etc., might be siphoned into the drinking-water supply system. Another similar condition exists in almost every large commercial laundry establishment where the water supplied to the tubs is brought in through a bottom connection. In many cases this connection is used not only to supply a mixture of hot and cold water, but on emptying the tubs a portion of this pipe is used for the drain. Here again it is possible to siphon the contents of the tubs into the drinking-water supply system, and what is worse, a direct cross-connection is made with the sewerage system.

The recent strides taken in the development of air-conditioning equipment and its general adoption for use in public buildings has introduced another problem in cross-connections. This is in the scrubber, so called where the air introduced into the conditioning equipment is cleaned and, humidified by spraying water against a baffled screen made of fine mesh or absorbent material. A steady flow of water is maintained during operation in warm weather and other times in order to provide additional moisture for the air so as to supply the humidity correction desired. In certain types of equipment, the excess water is recirculated by means of a small centrifugal pump with the suction made into the drip pan under the scrubber. The waste connection from the pan is connected with the sewer, and usually provision is made to supply water directly into the scrubber spray piping where it might become contaminated with water from the drip pan. Should the sewer overflow into the drip pan, there is the possibility that the water supply might be polluted with sewage.

The preceding conditions are those which might exist in any average community. However, as this subject deals with special equipment it might be well to describe a cross-connection that was found at a nearby airport whereby the water is used to dispense high-test gasolene for aircraft use. The equipment consists of several large underground storage tanks connected by means of piping to a pit in which is placed the coiled hose and control nozzle of a type familiar to all and now in common use at roadside filling stations. The idea in placing all of this equipment underground is to provide an unobstructed area in front of hangars at large airports. The underground tanks are filled with gasolene from tank trucks to within 5 or 6 in. of the top, after which the caps are securely fastened. A valve is then opened on the water-supply connection which is made to

the bottom of the tank, thus introducing water under city pressure. The gasolene, being of lighter specific gravity, floats, and as it is drawn off, an equal amount of water enters the underground tanks displacing the gasolene. This procedure continues until such time as an automatic float valve shuts off the water supply. This float valve is designed to keep the water-supply valve open when the float is in contact with gasolene, but when the water rises making contact with it, the float operates to close off the water supplied to the tank. The water in the tank is then wasted, in preparation for refilling the tank, usually into a sewer or other waste line. This in itself constitutes a cross-connection and to a certain extent introduces an explosive hazard in these waste lines. It can be readily seen that under the proper condition the gasolene might be drawn back into the water-distribution system. This would be unpleasant, to say the least.

A device has been placed on the market recently which is designed to replace the old method of pump spraying of trees, shrubbery, and gardens about the house by the less arduous task of directing a garden hose. This equipment consists of a special spray nozzle into which a cartridge of Bordeaux mixture, nicotine, arsenate of lead, or other "bug killer" may be placed. It is screwed onto a common garden hose. When water is forced through the cartridge, the solution is produced and may be used without effort. Should a vacuum occur in the water-supply piping, there is nothing to prevent the "bug killer" from entering the drinking supply.

This is not intended to be a complete listing of cross-connections found with special equipment. No doubt many of you have come in contact with several of these applications and know of others that are just as bad, if not worse. The general public and those in responsible positions at manufacturing establishments should be made acquainted with the hazardous conditions that these cross-connections present, and water department officials should be eager to do their part in their elimination.

TANKS FOR WATER SERVICE IN HIGH BUILDINGS.

BY H. E. HALPIN.*

[*Read November 19, 1936.*]

The purpose of this paper is to discuss those structural features and piping connections of tanks used for public water service in high buildings as far as these features have a bearing on public health.

Where the public water service cannot be made sufficient in pressure to serve the needs of high buildings, it is necessary to resort to pumping, usually in combination with storage either at grade or elevated.

I. Storage at Grade or Below Grade.

1. *Receiving Tanks.* As referred to in this discussion, the receiving tank is a small tank of steel or wood located at grade inside a building and of 500 to 5 000 gal. capacity. The following requirements for construction of receiving tanks have been drafted by the Chicago health and water authorities and it is expected will be incorporated into the Chicago Building Code which is in the final stages of revision. "The (receiving) reservoir shall be of ample size adequately to supply the pumping equipment. It shall be open to atmospheric pressure at all times. It shall be provided with an overflow pipe or pipes having a total area equivalent at least to one pipe size larger than the total area of the inlet pipe or pipes. The overflow and drain pipes shall not be directly connected to any sewer or drain. Vent pipes shall terminate downward. The vent and overflow pipes and any other opening in the tank shall be screened to prevent entrance of insects or rodents. The receiving reservoir shall have a self draining approved cover and be located where there is no exposure to pollutional hazards." These requirements are excellent for receiving tanks but should not be applied in full to all types of storage tanks. For instance, it is common practice to provide no special vent on elevated tanks, and frequently overflow pipes on outdoor above-ground tanks are not larger than fill pipes.

2. *Suction Tanks.* As referred to in this discussion, the suction tank is a sizable tank, say of 10 000 gal. or greater, of steel or wood, located at grade, either inside a building or outdoors. It is customary to cover outdoor tanks. Many indoor tanks are without covers. To avoid contamination from overhead drain or sewer piping, as well as other exterior sources, every suction tank should have a substantial, tight cover, with vent so screened and arranged as to preclude entry of foreign matter. Where water is being drawn continuously, it is customary to fill through a ball float with inlet above the overflow level so that no back flow or siphonage

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is possible. Where the suction tank is reserved for fire protection, the fill pipe is controlled manually. Overflow pipes from such tanks should discharge to atmosphere or through an air gap into a drain, and, under no conditions should be directly connected to sewers, as contamination could be caused by the flooding and backing-up of sewers.

3. *Cisterns and Reservoirs.* Cisterns and reservoirs below grade are used when space is not available for suction tanks. This type of storage is not readily accessible to complete inspection and is open to suspicion because of the masonry construction below grade. Cisterns and reservoirs for the storage of potable water should be constructed with tight masonry walls and bottoms so that ground water cannot enter; where outside of buildings, walls should be extended at least 6 in. above grade so as to exclude surface water, and roofs should be preferably of concrete. In any event, they should be watertight with a screened vent to atmosphere. Where the reservoir roof forms the floor of a building a raised manhole with self closing cover or other suitable entrance should be provided to avoid the entrance of surface wash. No return water of any description should be allowed to enter cisterns and reservoirs or any tanks used for public water service in high buildings except possibly from closed circulating systems where there is no possibility of contamination.

II. Elevated Storage for High Buildings.

You are all familiar with the steel or wood tank located on a short tower above the roof of a building or on an independent tower adjacent to a building. These tanks and towers must necessarily be of substantial construction to carry the large weight of water and to withstand severe winds. The outdoor tank practically always is built with a tight roof and if this is well maintained, there is little possibility of contamination from external sources. Many tanks located in pent houses have no covers. A substantial, tight cover should be provided ordinarily, but if the tank room is used for no other purpose and entered by only a few responsible employees, a wire screen would seem to be sufficient.

Filling of elevated tanks is usually through a by-pass around a check valve in the tank discharge pipe. In some installations there is no such check valve, the tank simply floating on the pipe system and being filled through its discharge pipe by intermittent pumping. In some cities it is specified that elevated tanks shall be filled through a line separate from the tank discharge pipe. From the public health standpoint, there appears to be no additional safety provided by this method, assuming that the piping arrangement is otherwise safe.

In New York City, it is required that an elevated tank "shall be fed through a separate line at least 2 in. in diameter, discharging into the top of each tank above the overflow level through the side or through the

bottom, that portion of the pipe within the tank to be of brass or copper or other non-corrosive material."

No contamination would be expected at overflow pipes of elevated tanks as either a stub overflow is used or the overflow is piped down to an open discharge at roof level or grade.

III. Pressure Tanks.

These are closed steel tanks partly filled with water and partly with air under pressure. They are used with automatic sprinkler equipments in high buildings in New York, Chicago, and a few of the large cities, and also in combination with automatic pumping on small domestic water systems. About the only possibility of contamination would be in the drain connection and, as in other types of tank installations, drains should not be connected to sewers. In practically all of the installations that the writer has seen, there has been a drain with open discharge from the pressure tank to roof level.

IV. Process Water Tanks.

All of the preceding discussion has been confined to tanks for the storage of potable water. Where a potable public water supply is used to supply a suction or elevated tank that is also supplied with an unapproved water, the tank should be open to atmospheric pressure and the potable water should be supplied above the maximum level of water in the tank.

V. Maintenance of Tanks and Reservoirs.

In addition to maintaining storage tanks in good structural condition, it is essential that they be emptied and cleaned periodically. A thorough cleaning is needed after repairs are made. Steel tanks used for fire service are scraped and painted internally about every five or six years. To the writer's knowledge, no well defined practice has been established for the periodic cleaning of water tanks and reservoirs in high buildings, such maintenance being left to the property owner. It is suggested that the Committee consider this matter. Cleaning of tanks should include adequate disinfection with fresh chloride of lime, H. T.H., chlorine, or other means. It is suggested that the Committee indicate the residual chlorine in parts per million or the amount of disinfecting agent that is considered necessary for proper disinfection. When the condition of water in storage tanks is in doubt, samples should be taken for bacteriological examination.

Conclusions. From a public health standpoint, the structural condition and piping connections of water tanks in high buildings will be found

reasonably good, but undoubtedly improvements are needed, as in other parts of interior piping systems, adequately to safeguard the water supplies in buildings and in street mains from contamination. Many faulty conditions at tanks could be corrected by inspection and proper notification of the building owners by the water or health authorities or other department having jurisdiction.

ELECTRIC GROUNDING ON WATER PIPES.

BY J. O. R. COLEMAN.*

[*Read November 19, 1936.*]

The question of grounding of electric light and power circuits has received attention almost from the beginning of the use of electricity for light and power. The original Edison System as installed in New York City was a direct-current system, and the practice was established to maintain circuits completely isolated throughout the customer's premises. This practice is still followed today in direct-current distribution systems wherever they are used.

In the beginning of alternating-current distribution systems, the practice of direct-current systems was followed. However, difficulties arose, and grounding the secondaries on the customer's premises appeared to offer the most practical way of eliminating the hazard to life and the danger from fire due to the high voltage that may be impressed by a cross between primary and secondary wires or a breakdown of transformer insulation. The National Electrical Code in 1901 permitted the grounding of such circuits and followed this in 1903 with a provision recommending that such circuits be grounded. However, it was not until 1913 that the Code made grounding mandatory for all 110–220 volt lighting circuits.

In the early days, many different forms and types of grounding were used. Pipes or rods were driven into the ground, plates buried, circuits connected to water pipes, to gas pipes, and to any other metallic object that might make an appreciable contact with the earth. Experience showed that water pipes afforded the most reliable and most satisfactory ground to accomplish the purposes for which grounds were made.

The reason why a continuous metallic public supply water piping system makes the best ground is not because such pipes have water in them but because such a piping system has by far the largest contact surface with the conducting material of the earth. In fact, where such a piping system exists, it is the earth, electrically speaking, for all practical purposes. The hazard to be protected against arises from the fact that voltages may accidentally be set up between different metallic objects in the house and that persons may be subjected to these voltages. Water pipes appear at numerous points through the house and are interconnected or in contact with steam pipes, gas pipes, etc., so that they offer numerous chances for people to get in contact with them. If some other grounding electrode were employed instead of the water pipes, the voltages between electric wiring, apparatus, etc., and this extensive system of pipes might be much larger and, hence, the hazard much greater. The very presence of these piping

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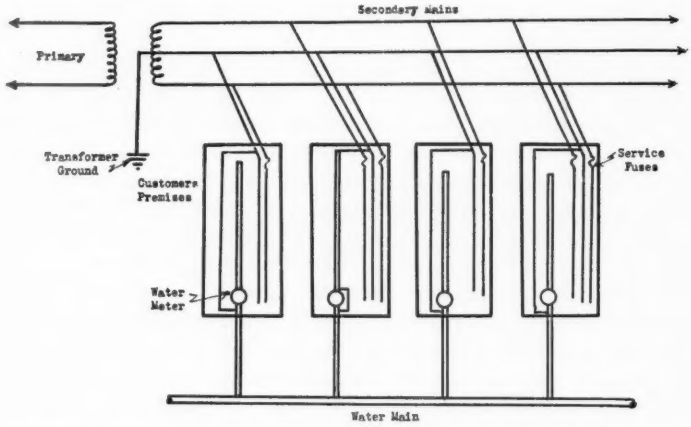
systems in a house increases the importance of using them as grounding electrodes.

There is shown in the accompanying diagram a typical layout of a secondary distribution system and a water main. The transformer supplying the electric energy is energized from a primary circuit at some voltage ranging from 2 300 to 13 200 volts. This voltage is necessary in order to deliver the power required from the point of supply. However, if allowed to remain on ordinary house wiring and fixtures, this voltage would prove dangerous to life and also would constitute a fire hazard. The transformer steps this voltage down to 115–230 volts. That is, a three-wire system where there are 115 volts from the center of each side and 230 volts between the



TYPICAL ARRANGEMENT OF ELECTRIC SERVICE WIRES AND WATER SERVICES.
For a single customer supplied by a transformer, the arrangement is the same as for the first customer's premises shown at the left of the diagram.

two sides of the circuit. The mid-point of this transformer is generally grounded to a driven ground at the base of the pole. For small consumers, a two-wire service is used, — that is, the neutral and one of the outside wires is carried to a customer. These two wire services are connected to the alternate sides of the secondary so as to obtain a balanced load on the secondary throughout and thereby obtain better voltage regulation. A large consumer, — that is, one who has a kitchen range, a water heater or a large house with a substantial amount of lighting, is supplied with a three-wire service. Today the neutral wire is grounded to the water pipe on the street side of the meter or, where this is impractical, to the house side of the meter and a bond wire placed around the meter. This neutral wire is not fused, either for the main service or for the individual branch circuits. The outer wire, on the other hand, is fused both for the main service and for the individual branch circuits. Lighting circuits are generally fused at

15 ampères. The circuits for power and for heavy duty appliances are, of course, fused more heavily so as to take care of the load that they carry. In an ordinary residence, the kitchen range and the water heater will frequently have 60-ampère fuses. The main service fuse will seldom, if ever, exceed 100 ampères.

With this setup we provide maximum safety on the customer's premises. The potential throughout the customer's premises is definitely limited to the 115 volts of the supply. In case of a contact between primary and secondary, or of a transformer breakdown, the potential within the premises is limited and a positive path is provided for fault current promptly to deënergize the circuit. Likewise, for a short circuit on the house wiring, a positive path is provided, and the secondary fuse clears the circuit.

Let us see what might happen when things are not quite normal. With this arrangement and the water meter removed we still have the same protection against the primary voltage both by the ground within the customer's premises and by the ground on other customer's premises. However, if the water meter is removed and the water system not grounded through other piping systems within the building, the secondary can short circuit to the water pipe. In this case there would be the full secondary voltage across the opening for the water meter. The possibility of this combination of circumstances occurring seems remote. In case of a break in the neutral ahead of the ground connection, the electric service would still operate and the water service would carry the neutral current. Opening the service pipe would result in some potential across the opening. However, there is no potential at or across the meter.

With this arrangement, carelessness on the part of the power lineman in connecting the secondary service would result in a direct short circuit through a low impedance path, and such a condition would promptly be noticed due to heating of the service wire and also due to a substantial reduction in voltage on one side of the secondary.

The opening of a service pipe outside of the ground connection would result in a voltage across the opening equal to the voltage drop of the neutral conductor, a maximum of 2 or 3 volts.

The situation is somewhat different when we consider a single residence supplied from a secondary. In this case the ground on the water pipe insures the limitation of differences of potential within the residence, and also insures a path for primary current in case of breakdown. Also, the same protection is provided for a breakdown between the secondary wire and the water pipes within the house. In this case, however, a reversal of the secondary connection might occur, and if the driven ground at the transformer was of relatively high resistance, say 100 ohms, little current would circulate. This might go unnoticed for some time. Such an arrangement would result in the secondary voltage appearing across an opening in this circuit. This condition is limited to a relatively few premises, and with reasonable care in workmanship, such occurrences will be negligible.

Under a few conditions, water companies have found it desirable to place insulating joints between their mains and the service connections. The use of such joints makes little difference on the effectiveness of the grounding of the secondary services. The primary purpose of limiting difference of potential within the premises is still accomplished and with a number of such services, averaging 20 or 30 in parallel, a low resistance ground is obtained to provide protection against the primary voltage. This protection against primary voltage could, of course, be obtained by driving a rod of equivalent length to the service pipe in each premises, and using that rod as the ground connection. However, such practice would not give the protection against differences of potential within the premises, as the water pipes still form a metallic conductor throughout the premises.

In the early days of electrical development, considerable difficulty was experienced on all underground piping and cable systems due to stray direct current causing electrolytic corrosion. Because of this experience, many water purveyors first objected to the use of water pipes as grounds for alternating current secondary distribution circuits. However, after several years of discussion it was brought out that protective grounding to water pipe systems, where such were available, constituted the only effective practical safeguard to life and property and that such connections would carry large currents only during short and comparatively infrequent periods when abnormal disturbances of the electric system caused these ground connections to fulfill their protective purposes. Tests by the Bureau of Standards definitely showed that electrolytic action of alternating current was negligible compared with the effect of direct current. On such a basis the American Water Works Association in 1920 adopted a resolution approving the grounding of secondaries of lighting transformers on water pipes for protective purposes. Recently several water works operators have received complaints, either from their customers or employees, that they had received shocks from the water pipes, or that sparks had occurred when pipes were disconnected or removed. Also, in certain situations difficulties due to undesirable tastes or odors in the water were attributed to the practice of protective grounding. The theory has been advanced that superimposed alternating current accentuates the action of small direct currents. However, this theory is still quite nebulous and no reliable and complete experimental evidence of the effect is available. Unfortunately in practically all of these situations complete information has not been available. In a number of cases such evidence as was available indicated that the trouble was due to irregular conditions or practices that were in complete violation of the National Electrical Code.

In May 1935, the American Water Works Association rescinded its previous official sanction of the grounding of electric light and power circuits to water pipes. A discussion of this action brought out the need for mutually acceptable technical data on the effects of grounding on water pipes and all other possible means of obtaining the necessary protection

so that the electric service could be fully and safely enjoyed by our people.

As a result of this discussion the American Research Committee on Grounding has been formed, under the sponsorship of the American Water Works Association and the Edison Electric Institute. Mr. Warren of the Bell Telephone Laboratories, and Chairman of Grounding Committee of the National Electrical Code, was selected to be chairman of this committee. In addition to the principal sponsors, the American Gas Association, American Institute of Electrical Engineers, American Society of Sanitary Engineers, the Telephone Group of the American Standards Association, the American Transit Association, Copper and Brass Research Association, International Association of Electrical Inspectors, National Association of Master Plumbers, the National Bureau of Standards, the National Fire Protective Association, and the NEW ENGLAND WATER WORKS ASSOCIATION are coöperating in this endeavor.

The committee held a meeting, organized and approved the following as the scope of its activities.

"To investigate and report upon, as conclusively as may be: (a) electrical grounding connections, made to water, gas or drainage pipes, with respect to the effects of any kinds of electrical current thereby caused to flow, upon (1) the pipes, (2) the contents of the pipes, and (3) fire and personal hazards; (b) conditions in the electrical circuits involved which produce current flow over pipes; (c) measurement of stray currents; (d) efficiency of electrical grounding."

A Technical Subcommittee has been appointed, and this subcommittee has been at work since that date. We naturally started by reviewing the overall problem, looking towards finding some general solution. However, such a research program seemed beyond the means of the committee as many years would be required to carry through an exhaustive program. The Technical Subcommittee, therefore, proposed that it start its work by investigating actual cases where grounding appeared to have any adverse reaction on the water system. The Committee believes that by the investigation of a number of specific cases the important features of the problem will be more definitely brought out and that any research activities will be directed into the most protective channels. C. F. Meyerherm, the Secretary of the Committee, has written a number of water purveyors requesting that they bring to the attention of the Committee any cases of difficulty that they believe are due to electrical grounding. I am sure Mr. Meyerherm will not object to my extending that invitation to the members of this ASSOCIATION. The Committee is anxious to obtain the facts,—that is, all the facts, so that the best overall solution can be obtained.

UNDERWRITERS' TESTS OF TRANSITE PIPE.

BY CHARLES W. SHERMAN*

[*Read March 18, 1937.*]

I am presenting to the New England Water Works Association, Report Number Ex. 1251 made by the Underwriters' Laboratories Incorporated on Transite Pressure Pipe and Couplings. The report is dated January 20, 1937. As the result of the tests and investigations covered by this report, the Underwriters' Laboratories Incorporated has approved "Class 150" Transite "Pressure" Pipe and Transite "Simplex" Couplings for use in public and private waterworks and in fire-service systems and connections to such systems where the working pressure does not exceed 150 lb. per sq. in.

Of the tests and investigations covered by the report, I have no personal knowledge except that which I have secured by examining its contents. It is felt that the presentation of the information included will be of value to our members.

I quote from the report:

Summary. From the conclusions drawn it will be noted that it is practical to handle and ship the pipe and couplings and install and maintain them in underground piping systems; that systems constructed of them are not subject to rapid deterioration; that the pipe and couplings are capable of withstanding all reasonable stresses to which they are likely to be subjected under ordinary service conditions; that they are reliable in service; that they are uniformly made and can be uniformly assembled.

Conclusions. The following conclusions apply to Transite Class 150 Pressure pipe and couplings in sizes from 4 to 24 in., inclusive, for use in underground water service where the working pressure does not exceed 150 lb. per sq. in.

Practicability. The pipe and couplings can be handled and shipped by the methods employed without excessive damage or breakage. They can be properly installed under conditions ordinarily met with in service, by those reasonably familiar with the installation of underground pipe; and repairs and renewals of Transite pipe lines can be accomplished without unusual difficulty.

The Service Record investigation also shows that the pipe and couplings can be handled, shipped, and installed without breakage or excessive damage under customary practices, and the fact that large users of this material are consistently sending in new orders indicates that the pipe and couplings already laid are giving good service.

Observation made during the tests at the factory and during field inspections of several Transite pipe installations show that it is practical for those reasonably familiar with underground pipe work to place this pipe in proper position in the trenches and assemble the joints. These observations also show that Transite pipe lines can be assembled with the ordinary tools provided, that they can readily be connected to existing cast-iron or steel water lines and that they can be installed under all conditions under which other types of underground pipe can be installed.

*Partner, Metcalf & Eddy, Boston, Mass.

The results of the Tapping Tests show that if a reasonable amount of care is exercised, it is practical to drill and tap directly into the wall of the pipe for service connections and that such connections are free from excessive leakage.

Durability. The Transite pipe and couplings dealt with in this report, when installed according to the methods advocated by the manufacturer, will probably withstand deteriorating influences resulting from exposure to potable waters and ordinary ground conditions for long periods of time.

It is generally recognized that ordinary Portland Cement products are attacked and disintegrated in relatively short periods of time by soils and waters containing soluble sulphates and weak acids. The manufacturer claims that the rate and extent of such disintegration is largely controlled by the free lime content of the composition and that Transite pipe differs from ordinary Portland cement products in that it has a very low free lime content and, therefore, has substantial immunity to such disintegrating agents.

The results of the investigation of the chemical properties of Transite pipe show this product to differ from ordinary Portland cement compositions in that the pipe contains a much lower percentage of free lime and a correspondingly higher percentage of silica.

The information relative to the durability of the pipe and couplings obtained during the tests and Service Record investigations shows that this pipe is not unduly susceptible to breakage in handling, transporting, and installation, and that it may be expected to withstand without undue breakage the stresses likely to be met with in the service for which the pipe and couplings are intended.

Experience has shown that rubber used in underground joints has given long and satisfactory service, the life of the rubber being favored by the dampness and darkness conditions.

Reliability in Service. When properly installed, Transite pipe and couplings will withstand without failure, the stresses to which they will ordinarily be subjected in service. Due to the smooth glossy interior surface, the friction loss in this pipe is low and is likely to remain low indefinitely.

The pipe and couplings which have been in service have not developed any defects which would indicate that they are not reliable under the conditions met in ordinary mains for underground water service.

Strength. Transite pipe and couplings made in conformity with the specifications for its manufacture and installed according to the methods advocated are capable of withstanding the working pressures for which they are designed and all stresses to which they are likely to be subjected in service.

The results of the Bursting and Tapping Tests show that the pipe and couplings have a minimum factor of safety of four, based on the maximum rated working pressure of 150 lb. per sq. in. While this factor is somewhat lower than in other types of listed pipe, it compares favorably with that required in iron body gate valves larger than 6 in. designed for the same working pressure.

The Service Record shows that Transite pipe and couplings used in underground water service in a number of municipalities have proven reliable in service and none of the replies received indicated that this material was lacking in strength under the service for which it was used.

The results of the Impact Tests show that the pipe is not unduly susceptible to breakage from impact.

Uniformity. The facilities and methods employed in the manufacture of the pipe and couplings which are the subject of this report are such as to insure an accurate

and uniform product. The uniformity with which the pipe is assembled under practical conditions in the field will be dependent upon the experience of those doing the installation work, the extent to which the instructions for installation and handling are complied with, and the thoroughness of the tests and inspections after the pipe and couplings are laid.

The examinations showed that the design of the pipe and couplings is such that the pipe can be uniformly and easily assembled. The Service Record investigation showed that persons accustomed to handling and laying underground pipe can handle and lay Transite pipe in a uniform manner without encountering any difficulties other than those common to other kinds of underground pipe.

Observations during the preparation of the test samples and during the tests show that the pipe and couplings can be made uniformly tight at pressures considerably in excess of the working pressures.

OUR INTRODUCTION TO TRANSITE PIPE.

BY HAROLD L. BRIGHAM.*

[*Read March 18, 1937.*]

Our Introduction to Transite Pipe took place in the summer of 1936 through a W.P.A. project for the installation of 2 000 ft. of 6-in. main in the outlying section of the city. The road on which the main was to be laid was very narrow and winding, with steep slopes on each side, and the digging we found, much to our sorrow, was practically all ledge, varying from rotten pudding stone to the hardest trap-rock. The first 300 ft. offered good digging in hard-pan, but after that we ran into hard ledge.

When the pipe was delivered, it was decided that it would be better to have it placed in stock-piles, rather than to lay it along the road, due to the chance of breakage by flying rock in the blasting operations. As there were no houses near-by, it was not necessary to cover the blasts very heavily.

A stock of 500 ft. of pipe was assembled at the beginning of the job, a stock of 1 000 ft. in the middle, and one of 500 ft. at the end, so that the maximum carry was not more than 500 ft. Due to its light weight, two men easily carried the pipe on their shoulders from the stock-piles to the point of installation in a short time. (See Fig. 1.)

The day before we were ready to lay the first pipe, the Johns-Manville Installation Instructor arrived on the job and carefully went over all the details with the constructing foreman, as neither the foreman nor any of the laborers on the project had ever seen or laid this type of pipe before, although all had been employed on pipe-line construction for the department for more than a year. The use of the puller was demonstrated and also the method of blocking under the pipe. As tests were to be made for leakage at the end of each day's run, a short section of pipe with an ordinary solid cast-iron sleeve was connected to the end with a compound joint, and in the end of the sleeve a cast-iron plug which carried a ¾-in. curb cock was installed together with its compound joints. The short section was easily put on at the end of the day's run. After properly bracing it, to take up the end thrust, and weighting it down, to off-set the upward thrust, the water was turned on, and inspection of each joint was made by the foreman and the Johns-Manville Installation Instructor.

In back-filling, the entire line was tamped with railroad tamping bars, under and up to the horizontal diameter, except at the couplings, and extreme care was taken that no stones touched the pipe. Then mounds were made over the center of the pipe nearly to the surface to withstand possible upward thrusts during the tests. These mounds were broken

* Superintendent, Water and Sewage Commission, Marlborough, Mass.

down and leveled after the tests, and the material was hand-tamped or puddled with water according to the conditions met. As much of the excavated material was very rough, selected material from the job was first used for bedding around the pipe in the ledge, but this was abandoned in favor of sand, as a sand-pit was situated near-by. (See Fig. 2.)

On the first day that pipe was laid, and by the way the excavating was all completed in this section, twenty-two men were employed on the line,

FIG. 1.— HANDLING TRANSITE PIPE IN TRENCH.

four pipe-layers and eighteen back-fillers. A line of 500 ft. of pipe was laid, a third of the trench was back-filled, and the line tested in 6 hours and 30 minutes.

In making the first test, the test pressure was 120 lb. per sq. in. at the start. The gate was cracked slightly, and as the pressure built up, a slight sideways movement of the test end was observed. This was overcome by the use of trench-jacks as braces. Twenty-seven minutes were required to fill the line and reach the full pressure. As the end of the line was about

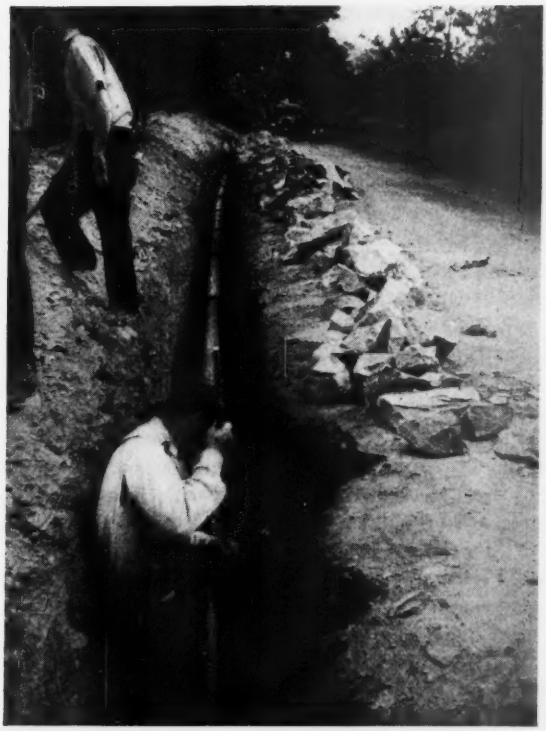


FIG. 2.— TAMPING UNDER TRANSITE PIPE.

FIG. 3.— TAPPING TRANSITE PIPE.

12 ft. lower than the beginning, a hose line was laid up the bank and over a wall so that the open end of the hose was higher than the inlet of the pipe. No evidence of air was observed, and no leakage at any of the couplings was found.

On the next day that pipe was laid, the trench ran through nearly all solid ledge, and 400 ft. of pipe were laid, the trench was partly back-filled, and the line tested, all between 9 A.M. and 2.30 P.M. or in five working hours. This pipe was bedded-in and covered with sand. When the test was made, one of the collars was found to be weeping slightly on the end. Apparently it had a slight crack. This collar was removed, replaced, and tested in thirty-five minutes. Examination showed that it had been cracked from a blow of some sort, probably having been dropped in a stone pile.

Several pieces of pipe were found to have dents in their sides, and three of these were rejected as we did not wish to take any chances. The indentations were about 2 in. long and $\frac{1}{8}$ in. deep and occurred about 52 in. from the ends. Apparently they were made before the pipe was cured, and it looked as though a rope or wire cable had pressed against them. Tests were made of these pieces of pipe and they withstood a 600-lb. per sq. in. hydrostatic test with no indication of failure at 780 lb. per sq. in., a factor of safety of 5.2 for Class 150 pipe.

On the last section laid, the usual tests were made and everything was apparently satisfactory at 4.30 P.M. Therefore, the pressure was allowed to remain on the line. About 6 P.M. one of the couplings broke horizontally on two sides. The trench flooded and the line shut off. This break was probably due to a crack received either in transit or in handling, although it was not visible when installed. When work was resumed on the following morning at 7.30 A.M., the trench was practically dry, except at the point of the break, and only a small amount of pumping was necessary to dry out the trench and replace the broken coupling. This took a little less than an hour and a half.

One hydrant connection was made in the middle of the line and one at the end, using a cast-iron globe fitting and a cast-iron pipe for the hydrant branch. Short pieces of pipe about 3 ft. long were used on each side of the gate installed on the main line at the middle point. Two service taps were made with a standard tapping machine on the bank before the pipe was installed. (See Fig. 3.)

No other construction troubles developed, but several days after the job was completed, complaints were received of a lime or cement taste. This was eliminated by the use of a small bleeder on the hydrant at the end of the line.

On the whole, the installation was satisfactory, but time alone will tell what troubles may develop.

EXPERIENCE WITH TRANSITE PIPE.

BY A. B. RICH.*

[*Read March 18, 1937.*]

During the last few years Transite pipe has been gradually growing in popularity for use in water mains and has successfully entered into competition with cast-iron pipe. Transite pipe, made by the Johns-Manville Corporation, is a seamless pipe made of asbestos fibre and Portland cement. The asbestos fibre is somewhat analogous in its effect to the steel in reinforced concrete. The toughness and tensile strength of Transite pipe are due partly to the reinforcing effect of the asbestos fibres.

The tensile, compressive and shearing strength of Transite is naturally not as high as for cast iron, but by making the shell of the pipe sufficiently thick it can and has been used successfully in the place of cast iron for water mains. Even with this thicker shell, the pipe is considerably lighter in weight than cast-iron pipe.

The joints of Transite pipe are made with Simplex flexible couplings which consist of a transite sleeve and two rubber rings. The space provided between the outside of the pipe and the inside of the sleeve is designed to compress the rubber rings to approximately one-half of their original thickness. This provides sufficient friction to resist movement of the rings after the joint is assembled. These joints are easily made and are practically water tight. It is general practice to use cast-iron fittings in Transite pipe-lines. These fittings should be all bell-ended, with over-size bells for sizes over 8 in. Leadite makes a satisfactory joint where Transite joins cast-iron fittings or gate valves.

In the summer of 1935, a complete water system was built at Sterling, Mass., under the supervision of Fay, Spofford & Thorndike with the writer as Resident Engineer. This was a P.W.A. project. The system consists of a driven well system, pumping station, standpipe, distribution mains and appurtenances. The supply is obtained from the driven wells and pumped into the standpipe and distribution mains by means of a centrifugal pump. Transite pipe was used with cast-iron fittings for the water mains. (See Fig. 1.)

Experience with Transite Pipe at Sterling, Mass.

Laying the Pipe. Transite pipe can be laid much faster and more easily than cast-iron pipe. The joints can be made satisfactorily in either wet or dry trenches. Difficulty is sometimes experienced in pulling a Simplex coupling into place if trench water containing small particles of silt or clay

*With Fay, Spofford & Thorndike, Consulting Engineers, Boston, Mass.

comes in contact with the pipe barrel. These small particles of clay tend to cause the rubber rings to slip, thus preventing the joint from being made properly. This trouble is remedied by washing off the ends of the pipes with clean water just before assembling the joint. Expansion is allowed for in the pipe line by allowing $\frac{1}{8}$ to $\frac{1}{4}$ in. clearance between the butt ends of the adjoining pipes. This is provided for immediately after each pipe is laid by raising the free end about knee high and then lowering it again into place. If Transite pipe is laid below the ground-water level, it must be weighted down by either partially backfilling the trench or by other means, as the pipe will float unless filled with water.

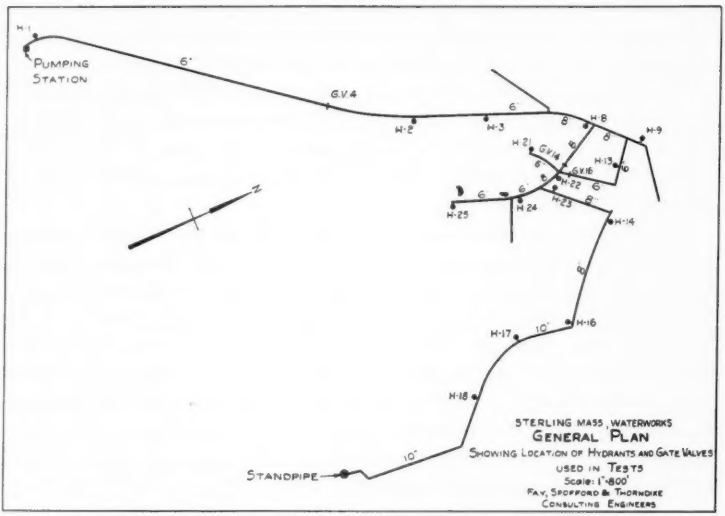


FIG. 1.— GENERAL PLAN — STERLING, MASS., WATER WORKS.

Backfilling. Considerably more care must be given to the backfilling of Transite pipe than is generally required with cast-iron pipe. Railroad ballast bars are very satisfactory for filling and tamping underneath and around the pipe. Care must be taken not to allow stones of appreciable size to drop on the pipe, as they are very apt to cause a localized rupture of the pipe. These localized ruptures are in the form of small indentations at the points of impact between the stones and the pipe and are not easily detected by inspecting the outside of the pipe. They are, however, readily detected by looking inside the pipe, as the ruptures will bulge out like a blister.

Backing-up of Pipe Lines for Unbalanced Forces. Bends, tees, dead ends, etc., must be backed up or shored up to take the full unbalanced

forces that may come upon them. Practically no resistance is offered to these forces by the flexible joints, and this construction detail cannot be neglected. We have laid 6-, 8- and 10-in. Transite pipe in gravelly soil on curves with radii as small as 125 ft. without special backing, and experienced no difficulty with lateral movement. Of course in soft soils like peat, etc., such a small radius would not be advisable without taking special precautions against lateral movement.

Service Connections. Since Transite is of dense, close texture, it can be readily tapped with the same tools and in the same manner as iron pipe, and the pipe itself can be cut by an ordinary wood saw. At Sterling, taps for 125 service connections were made, and no difficulty was experienced.

Leakage. Transite pipe lines give a good water-tight job. At Sterling, we made leakage test of about 19 000 ft. of 6-, 8- and 10-in. pipe under pressures ranging from 80 to 175 lb. per sq. in., and found that the Simplex couplings made water-tight joints even under relatively high pressures. Out of 1 839 pieces of Transite pipe tested, no single length showed any signs of sweating under pressure. Also out of 1 607 Simplex couplings tested only two were found to leak. One was due to a rubber ring slipping in pulling the joint, and the other was caused by a deep longitudinal scar on the pipe at the ring. This was probably caused by dragging the pipe over the ground. The results of these leakage tests are tabulated at the end of this paper.

Value of C as Used in the Hazen-Williams Formula. Transite pipe has a high value of C as used in the Hazen-Williams formula. In several experiments made at Sterling, the value of C ranged from about 145, for pipe lines containing several bends and special cast-iron fittings, to about 150, for straight lines containing few special cast-iron fittings. The results of these tests are tabulated at the end of this paper.

Water Hammer. Several tests for water hammer were made at Sterling. The results showed that the water hammer, or surge in the pipe lines due to changing rates of flow, was about the same as for cast-iron pipe lines. The results of these tests are tabulated at the end of this paper.

Strength. In the design of Transite pipe, extra thickness of shell to resist corrosion is not provided by the manufacturers as is generally done by the manufacturers of pipes made of corrosive material.

At Sterling, we tested some 19 000 ft. of 6-, 8- and 10-in. Transite pipe, after the trenches had been backfilled, at pressures ranging from 150 to 265 lb. per sq. in. During these pressure tests, several lengths of pipe burst under the higher pressures, but after replacing these defective lengths no further trouble was experienced.

Results of Leakage Tests.*

Test No.	Length of Pipe Tested, Ft.	Size, Inches.	Number of Simplex Couplings			Number of Leadite Joints			Pressure** Used in Test, Lb. per Sq. In.	Actual Leakage in G.P.D. per Mile, Per In. Diam.	Remarks
			6-in.	8-in.	10-in.	6-in.	8-in.	10-in.			
1	4170	6	416	...	...	33	...	...	175	43.1	
2	4610	6	460	...	...	38	...	...	175	91.5	
3	6073	6	546	...	...	65	...	...	...	...	
	1152	8	...	84	...	...	20	...	160	200.0	One leadite joint leaked and later fixed.
4	7290	6	724	...	...	89	...	...	...	...	
	2234	8	...	174	...	...	21	...	155	111.0	
5	8270	6	818	...	...	108	...	...	...	...	
	2247	8	...	174	...	...	23	...	163	93.5	
6	9360	6	929	...	...	125	...	...	...	...	
	3140	8	...	248	...	...	39	...	153	34.4	
7	9400	6	933	...	...	139	...	...	...	...	
	4590	8	...	358	...	...	52	...	150	56.6	
8	1083	10	...	...	80	...	...	4	...	...	
	20	6	1	...	...	4	...	...	...	...	
	33	8	...	4	...	...	5	...	80	165.0	
9	9486	6	944	...	...	156	...	...	...	...	
	4683	8	...	380	...	...	61	...	...	...	
	3586	10	...	...	283	...	...	24	125	77.8	
10	9486	6	944	...	...	156	...	...	...	...	
	4683	8	...	380	...	...	61	...	...	...	
	3586	10	...	...	283	...	...	24	80	35.0	Final Test.

*All Simplex couplings looked bottle tight. The leakage was apparently from the Leadite joints only. The Leadite joints were green as none of them had a chance to tighten up under continued water pressure at the time of the tests. There was no evidence of the Transite pipe sweating during any of these tests.

**These average pressures are given as the actual pressures in the pipe line varied according to elevation. The leakage is given in gallons per day per mile per inch diameter.

All Simplex couplings were left open for inspection during the leakage tests, and no leakage appeared at these joints.

Flow Tests for Determining the Value of C as Used in the Hazen & Williams Formula

Test No.	Hyd. No.	Feet of Pipe from Tank to Hydrant			Flow, G.P.M.	Pressures Lb. Per Sq. In.		Lost Head			Factor $\frac{A}{B}$	Actual Value of C	Remarks
		10-in.	8-in.	6-in.		Static	Residue	Lb. Per Sq. In.	Ft. A	C = 100 B			
1	24	3,585	2,390	228	....	92.5	57.6	35.0	80.8	151.4	0.534	140.2	Flow from H-25.
	25	3,585	2,390	913	1075	95.5	40.5	55.0	127.0	247.4	0.514	141.8	
2	22	3,585	2,811	....	....	92.1	77.2	14.9	34.4	67.57	0.509	142.0	Flow from H-1*.
	8	3,585	3,300	....	....	95.8	78.7	17.1	39.5	75.72	0.521	141.1	
	3	3,585	3,795	688	....	96.0	69.0	27.0	62.4	131.57	0.475	150.0	
	2	3,585	3,795	1531	....	96.5	57.5	39.0	90.0	189.97	0.474	150.1	
	1	3,585	3,795	5903	740	118.0	19.0	99.0	229.0	492.97	0.465	151.4	
3	3–1	.....	.....	5215	740	.....	.....	.....	166.6	361.0	0.461	152.0	Between H-3 and H-1 the pipe is straight and to an even grade.
4	16		.....	....	....	45.0	33.0	12.0	27.75	56.1	0.494	146.5	Flow from H-17.
	17	2,904	.....	....	1420	.....	.....	.....	.....	.....	.....	.....	
	18	2,029	.....	....	....	31.0	22.0	9.0	20.75	39.2	0.530	140.9	
5	14	3,585	1,383	....	1180	.....	.....	.....	.....	.....	.....	.....	Flow from H-14.
	23	.....	.....	....	....	100.0	80.0	20.0	46.2	103.5	0.446	154.7	
6	24	3,585	2,390	228	960	.....	.....	.....	.....	.....	.....	.....	Flow from H-24.
	25	.....	.....	....	....	100.0	76.0	24.0	55.5	122.8	0.452	153.6	
7	9	.....	.....	....	1240	.....	.....	.....	.....	.....	.....	.....	Flow from H-9.
	22	3,585	2,811	....	....	102.0	65.0	37.0	85.5	179.6	0.476	149.9	
8	2	.....	.....	....	....	102.0	22.0	80.0	184.3	383.3	0.481	148.4	Flow from H-3.
	3	3,585	3,795	688	1300	.....	.....	.....	.....	.....	.....	.....	†See Test No. 2.
	8	3,586	3,300	....	....	100.0	55.5	44.5	102.7	213.1	0.482	148.2	

*See General Plan; this parallel flow is converted into equivalent single 8-in. line of 412 ft.
†For method of calculating the value of C, see page 297.

SURGES IN PIPE LINES.

Test No.	Readings at Hyd. No.	Static Pressure in Lb. per Sq. In.	Surge Pressures in Lb. per Sq. In.		Flow From	Rate of Flow, G.P.M.	Time to Stop Flow		Remarks
			Min.	Max.			Min.	Sec.	
1	16	45	..	60	H-17	1420	1	30	
	18	31	..	48	H-17	1420	1	30	
2	2	102	..	165	H-3	1300	1	30	
	8	100	..	147	H-3	1300	1	30	
3	22	102	45	144	H-9	1240	1	30	
4	23	100	..	130	H-14	1180	1	30	
	24	101	..	136	H-14	1180	1	30	
5	25	100	..	170	H-24	960	1	30	
	22	102	..	146	H-24	960	1	30	
6	2	102	32	218	H-1	800	1	17	While water was flowing out of H-1 at a rate of 800-G.P.M. Gate valve No. 4 was closed in 1 min. 17 sec. The pressure at H-2 rose steadily from 50 lb. per sq. in. and then dropped back to a min. of 84 lb. per sq. in. H-1 was shut off and gate valve No. 4 was opened slowly. The pressure rose to a max. at 218 lb. per sq. in. at H-2 and dropped to a min. of 32 lb. per sq. in.
7	At the end of Pipe in Pump. Sta.	130	25	220	H-1	700	0	43	
8	1	124	80	180	....	105	Instantly		This is the normal surge in the pipe near the pumping station due to the stopping of the pump.

*These tests were conducted after the whole system had been subjected to the pressure tests required by the specifications and accepted. No trouble was experienced with the pipe lines due to these surges.

Failures from Pressure Tests (Summary)

Size of Pipe, Inches	Length of Pipe Tested, Ft.	Number of Couplings	Number of Lengths of Pipe	Pressure Range Used in Tests in Lb. per Sq. In.*	Length of Pressure Test, Minutes	Trouble Experienced before a Satisfactory Test was Obtained		
						Number of Pipe that Broke	Number of Couplings that Split	Number of Rubber Rings that Blew Out
10	3,586	283	293	152 to 186**	10	1	None	None
8	6,178	428	455	186 to 240	10	12	None	1
6	10,204	1,015	1,091	234 to 264	10	20	2	2

*The pressure range is given as the pipe was under different pressures depending on elevation of line.
All pipe used on project was Class C, designed for a working pressure of 130 lb. per sq. in.
All pipe was tested at 260 lb. per sq. in. before delivery to job.
All trenches were backfilled before pressure tests were conducted.
**The pressures used for breakage tests were purposely made sufficiently high so as to be sure that the pressure would not be exceeded under normal conditions. Does not cover pressure at breakage.

Out of 1 839 pieces of Transite pipe tested for breakage, 33 lengths failed; out of 1 607 Simplex couplings, two failed; and out of 3 214 rubber rings, three blew out. The results of these pressure tests are tabulated at the end of this paper.

After conducting the pressure tests, we concluded that a hydrostatic pressure test of 260 lb. per sq. in. for inspection at the factory is insufficient for Transite pipe for a working pressure of 130 lb. per sq. in. After making figures on the probable stresses developed in water mains in trenches and allowing for a suitable factor of safety, we concluded that Transite pipe should be strong enough to withstand a hydrostatic pressure of about 600 lb. per sq. in. before delivery to the job.

Physical Properties of Transite. Transite is an asbestos-cement product. The relative importance of the cement and asbestos fibre is difficult to determine. The water-cement ratio is kept down so that the amount of water in the mixture approaches within a few per cent. that actually needed for the proper hydration and crystallization of the cement. This reduces the initial shrinkage stresses in the material. The curing is carried out in accordance with the best methods of curing asbestos-cement products. The asbestos fibre appears to act as a binder for the cement and to aid in relieving shrinkage stresses as well as to increase the toughness of the material. The average ultimate strength in tension is about 3 500 lb. per sq. in., and the strength in compression about 6 500 lb. per sq. in. The modulus of rupture in crushing is about 7 000 lb. per sq. in. No definite figures are available on the shearing strength of the material. The modulus of elasticity is approximately 1 800 000 lb. per sq. in.

Transite apparently has a unique characteristic in that stress is very nearly proportional to strain up to 90 per cent. of the ultimate strength of the material. It appears to have no yield point and no well defined elastic limit. The material appears to be in a critical state when it is subjected to a stress above 90 per cent. of its ultimate strength. Let us assume as an illustration that the ultimate strength of Transite is 4 000 lb. per sq. in. This material can apparently be subjected to a large number of repeated strains for stresses up to 90 per cent. of 4 000 lb. per sq. in., namely 3 600 lb. per sq. in., without weakening the material due to fatigue. On the other hand, if the material were subjected to a tensile stress of 95 per cent. of the ultimate, it would be apt to become over-strained and result in failure if this high stress were held for a period of time.

In short, if the material were gradually stressed to rupture, it would fail at 4 000 lb. per sq. in. If it were stressed up to say 97 per cent. of 4 000 lb. per sq. in. and then relieved and later brought up again, the material might break at a stress lower than 97 per cent. of 4 000 lb. per sq. in. Transite should not be stressed beyond 90 per cent. of its ultimate strength as the material will be very apt to be seriously over-strained and thus cause the material to be weakened.

Advantages. Transite pipe has many advantages. It is lighter in weight and can be more easily handled than cast-iron pipe; it can be laid rapidly with fewer men in either wet or dry trenches. The leakage at the joints is almost negligible, thus giving considerably less waste of water than is generally experienced with other types of pipe. The pipe should not deteriorate with use due to tuberculation or diminished internal diameter, which generally occurs in iron pipe. Transite pipe not only has a high value of C as used in the Hazen-Williams formula when new, but should maintain its carrying capacity. The water conveyed through Transite pipe should not deteriorate in passing from its source to the consumer as is often experienced with iron pipes carrying soft water or water containing iron. Since the joints in Transite pipe lines are flexible, the pipe may be laid in places where unequal settlement is likely to occur.

At Sterling, no trouble has been experienced with the water at dead ends, thus eliminating the need for periodic flushing. In times of heavy draft, such as in the case of a fire, the water in the system does not become turbid with iron rust as is sometimes experienced with iron water mains.

Disadvantage. Transite pipe showed only one major disadvantage on this project; that was lack of sufficient strength to withstand rough treatment.

Conclusion. After weighing the advantages and disadvantages of Transite pipe we feel that this pipe has much merit for water main and will in the future become more and more favored for this purpose. It is an ideal pipe material, combining durability, freedom from tuberculation, corrosive and electrolytic ills, and it has sufficient strength to withstand the normal strains that come upon it in trenches.

Method of Calculating the Value of C in the Hazen and Williams Formula

Notation:

A = Actual measured loss in head in feet between two points on a pipe line.

B = Calculated loss in head in feet for $C = 100$ between the same two points for a given rate of flow.

$\frac{A}{B} = K$ = a convenient factor for determining the value of C that corresponds with A.

Formulae:

(1) $Q = av.$

(2) $v = 1.318\ CR^{0.64}S^{0.54}$ (Hazen & Williams Formula).

Combining (1) and (2) $Q = 1.318\ CaR^{0.64}\ S^{0.54}$.

For a given size pipe $1.318\ aR^{0.64} =$ a constant $= X$.

Then $Q = XCS^{0.54}$.

$$\text{Slope} = S = \frac{A}{e} = \left(\frac{Q}{X}\right)^{\frac{1}{0.54}}\left(\frac{1}{C}\right)^{\frac{1}{0.54}}$$

$$A = e\left(\frac{Q}{X}\right)^{\frac{1}{0.54}}\left(\frac{1}{C}\right)^{\frac{1}{0.54}}$$

$$B = e\left(\frac{Q}{X}\right)^{\frac{1}{0.54}}\left(\frac{1}{C}\right)^{\frac{1}{0.54}};\ \frac{A}{B} = \left(\frac{100}{C}\right)^{\frac{1}{0.54}} = K.$$

$$C = \frac{100}{K^{0.54}} = \text{the value of } C \text{ that corresponds to } A.$$

A graph of this equation can be plotted to aid in solving this type of problem.

Stresses in Water Mains.

An accurate analysis of the stresses developed in water mains due to trench loads is very difficult on account of the uncertainty of the loads, and simple analysis of stress accompanied with experimental data will be of more value.

A large percentage of the total stresses developed in the smaller sizes of water pipe is due to the external loads from the backfill. The external loads cannot be eliminated entirely by careful backfilling, and their full effect should be allowed for in the design of the pipe. Pipe tested before delivery to the job should be tested by an internal hydrostatic pressure sufficient to develop stresses in the pipe that would be at least equivalent to the maximum probable combined stresses that are likely to be developed in the pipe after it is put into service.

Suppose, for instance, that 8-in. Transite pipe Class 150 is to be installed for a working pressure of 100 lb. per sq. in. with allowance for a water hammer of 50 lb. per sq. in. If this pipe were tested for a pressure of 300 lb. per sq. in., or twice the maximum working pressure, it would not be sufficient to stress the pipe to a point that would be very likely to be reached in the pipe after installation. This pipe should be strong enough to withstand at least 600 lb. per sq. in. internal hydrostatic pressure to allow for a factor of safety and for super-imposed loads that are likely to come on it from time to time in service.

TRANSITE PIPE RIVER CROSSING AT SCITUATE, MASS.

BY FRANK A. MARSTON.*

[*Read March 18, 1937.*]

The Humarock section of the town of Scituate, Massachusetts, is a narrow neck of land lying between Massachusetts Bay on the east and the South River on the west. There is no land connection between this section of the town and the rest of Scituate, the two parts being separated by a tidal estuary at the confluence of the North River and the South River. For a distance of about three miles, the tidal South River forms the boundary between Scituate on the east and Marshfield on the west.

The water supply of the Humarock section of Scituate is obtained from wells in Marshfield and was formerly pumped by means of centrifugal pumps, across South River through a 3-in. cement-lined pipe, located on the Marshfield Ave. bridge. About four years ago (1933), it was deemed advisable to supplement this supply main with a larger pipe, in order to provide better protection in case of fire. It was decided that a 10-in. pipe should be laid under South River† about 1 000 ft. south of the bridge. This called for some 550 ft. of pipe to be laid between high-water marks, including about 180 ft. under water at mean low tide. (See Fig. 1.)

Description of Pipe and Joints. The soil in which the pipe was to be laid consisted of sand and gravel, mixed with saltmarsh muck. Because of previous experience with corrosion of the exterior of cast-iron pipe in such material, it was believed that Transite pressure pipe would be preferable to cast-iron pipe for this location. Transite pipe is manufactured from a mixture of asbestos fiber and cement. It is formed on mandrels and subjected to high pressure, a dense, homogeneous structure being thus produced, which is resistant to certain destructive agencies. Such pipe has been in use for a number of years in Great Britain, and in Italy.

The 10-in., Class D, Transite pipe used at Scituate was furnished in 13-ft. lengths, the outside diameter of the pipe being 12.04 in. At each end, the pipe was turned down to Class C thickness, 0.92 in., to form a shoulder for the rubber gasket. Each length weighed about 413 lb. without flanges. The latter, also made of Transite, were 2 in. thick with an outside diameter of 18 in. The flanges were pulled on over a rubber compression ring, to make a tight joint, and were bolted together with twelve $\frac{7}{8}$-in. galvanized bolts. (See Fig. 2). Each section of pipe with its flanges was filled with water and subjected to a pressure of at least 200 lb. per sq. in. on the ground

*Partner, Metcalf & Eddy, Engineers, Boston, Mass.

†W. J. Lumbert, Am. City, **48**:10, 54, Oct. 1933.

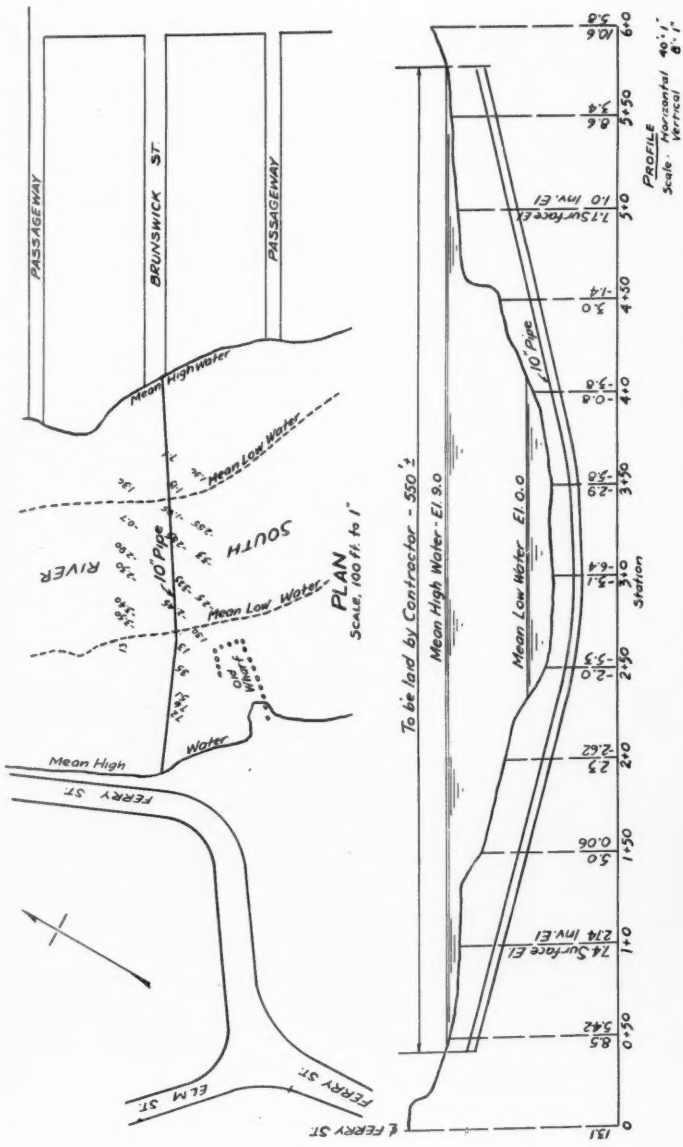


FIG. 1.— PLAN OF WATER MAIN CROSSING — SOUTH RIVER.

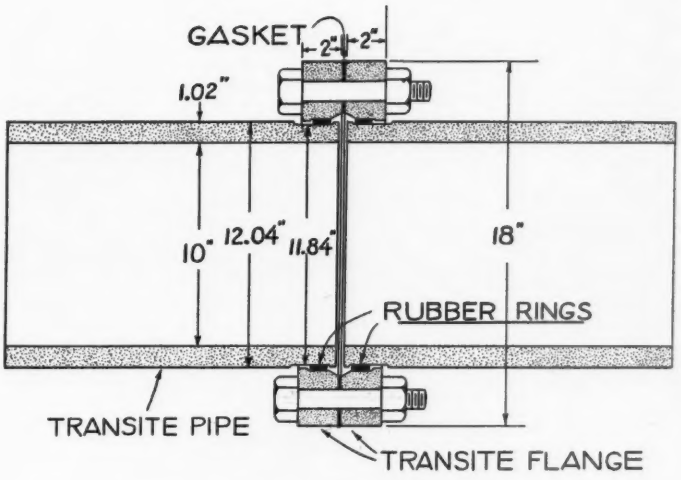


FIG. 2.— SPECIAL FLANGED COUPLING.

FIG. 3.— TRANSITE PIPE — SHOWING SPECIAL FLANGED JOINTS.

FIG. 4.— TESTING INDIVIDUAL LENGTHS OF TRANSITE PIPE TO 200 POUNDS PER SQUARE INCH PRESSURE.

FIG. 5.— FORWARD END OF TRANSITE PIPE ASSEMBLY.

FIG. 6.— TRANSITE PIPE ASSEMBLED ON SKIDS.

FIG. 7.— PIPE IN PLACE SHOWING SOUTH RIVER AND HUMAROCK BEACH IN BACKGROUND.

before being assembled on the skids. Special flanged joints, instead of the customary Simplex joints, were used, so as to develop a tensile strength in the pipe line as a whole. In connecting the sections of pipe, a service gasket was placed between the flanges, and the bolts were set up tight. The joints thus formed were flexible. The bolts and nuts were dipped in hot asphalt when the connections were made, in order to prevent corrosion of the only metallic portions of the assembly. After each flanged joint was made tight, the bolts and nuts were re-coated with the same material. (See Figures 3 and 4.)

Laying Pipe below Low-water Mark. That portion of the pipe line laid below mean low tide was connected up on shore and dragged into place as a unit by means of a cable, operated by a motor across the river. Before this process could be completed, it was necessary to excavate a trench in the river bed, not less than 3 ft. deep at any point. This was done by means of a drag bucket, operated from shore. It was found necessary to carry on blasting operations at a few points in order to remove rock. A diver was employed to place the explosive and inspect the results of the blasting.

Meanwhile, a wooden cradle of two lines of 2 by 4 in. spruce was built, approximately 300 ft. long. The cradle extended back from the shore, nearly in line with the trench in the river bed. Twenty-three lengths of Transite pipe were bolted together and securely fastened to the cradle. A blank flange was placed on either end of the assembled pipe, and a water pressure of 35 lb. per sq. in. was applied. The purpose of this was to test the joints for leakage and by separating the sections leave the pipe free to deflect and assume the required position in the river bed. Under this pressure the line lengthened about 6 in. (See Figures 5 and 6.)

After completion of the trench, the entire pipe assembly was pulled into place. Tackle was arranged in such a way that most of the pull was applied to front end of the cradle and the balance was applied to the far end of the pipe by means of a cable passing through the pipe. The end of the excavation was inclined in such a way that the pipe assembly entered the trench without flexing any joint up to its maximum permissible angle of 10 deg. The cable was hauled by a gasoline-engine-driven hoisting-drum. In order to minimize the whipping action of the cable and prevent damage to the pipe, rafts were placed over the pulling cable, to take up some of the sag. The leading end of the pipe (protected by a cast-iron companion flange) was open to allow the pipe to fill with river water as fast as it was pulled into place, in order to prevent flotation. The whole assembly, weighing about 7½ tons, was pulled slowly into place. (See Fig. 7.)

When the pipe line was first pulled into place, it was tested with too great a water pressure, and due to insufficient backfill and the flexibility of the joints, a section of pipe was deflected to a greater angle than it was designed to take, causing the pipe to be cramped at the flange and broken

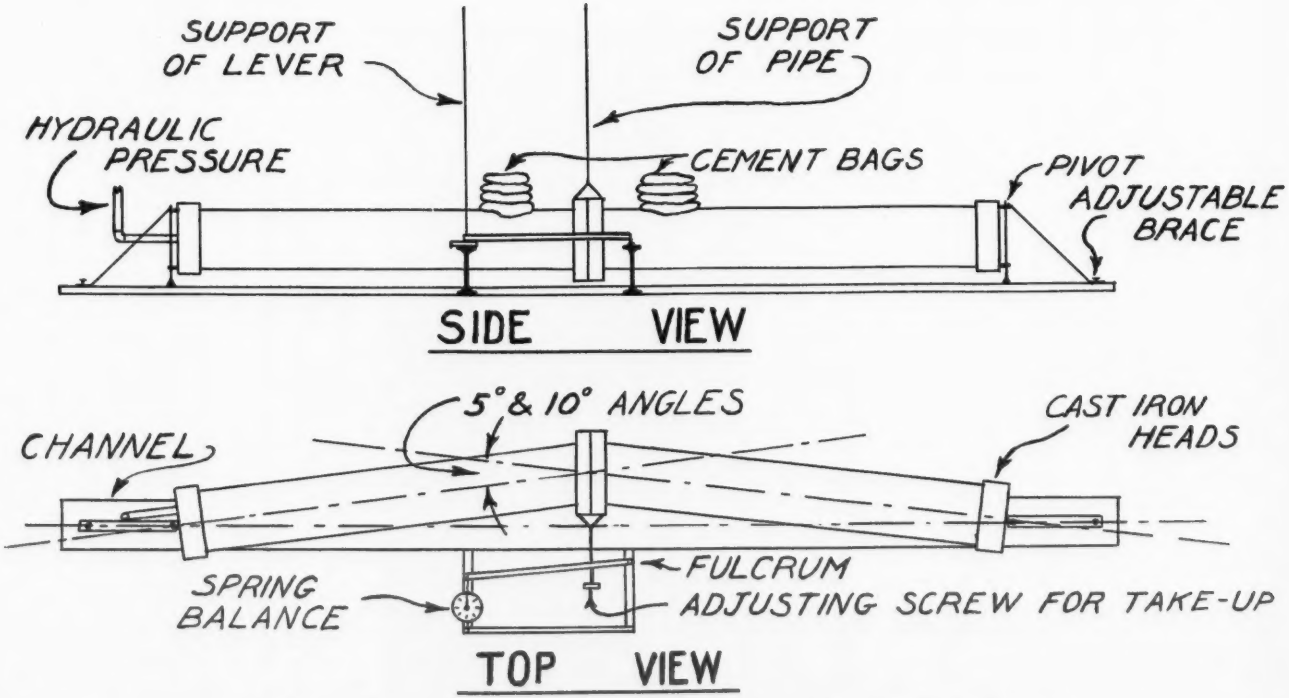


FIG. 8.— ARRANGEMENT OF PIPES FOR TESTING SIDE THRUST DUE TO DEFLECTION OF JOINT.

off. The pipe line and skid were then pulled out, each length of pipe tested to 200 lb. per sq. in., the broken pipe replaced and the whole line pulled back again into the trench.

Side Thrust in Flexible Joint Pipe. The Waukegan Laboratory of Johns-Manville Corporation conducted a series of tests to determine the force necessary to hold two lengths of 10-in. Class C pipe when deflected

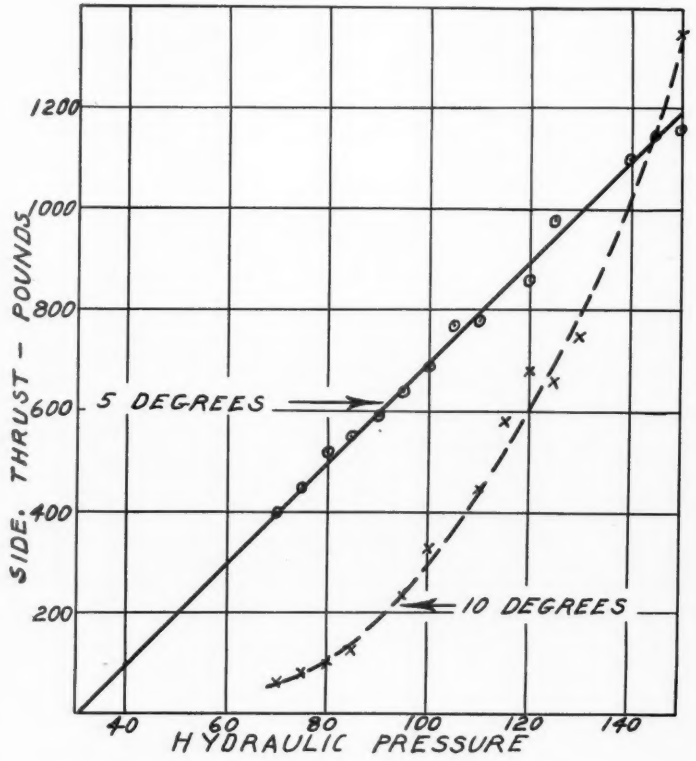


FIG. 9.— SIDE THRUST DUE TO DEFLECTION OF JOINT.

5 and 10 deg. The pipes were joined by a flange coupling of the flexible type herein described. The outer ends of the two pipes were fitted with screwed caps arranged with pivots so as to be free to turn in a horizontal plane. A hanger was provided to prevent the pipes from deflecting down, and cement bags were piled on top to prevent uplift. The side force required to hold the joint was measured by a lever and scale. The pipes were first set so that their center lines formed an angle of 5 and then 10 deg.

A maximum water pressure of 150 lb. per sq. in. was reached inside the pipes in each test. (See Fig. 8.) The resulting side thrusts in pounds for various water pressures and for each of two angles of deflection are shown in Fig. 9. The lower side thrust observed at 10 deg. deflection was probably due to some of the reaction being taken by the coupling.

Weighting the Pipe. As soon as the pipe was in place and had been checked for line and grade, the ends were blanked off and a pressure of 25 lb. per sq. in. was applied to test the joints for leakage. This test disclosed no leakage.

The next step was to place pre-cast blocks of concrete upon the pipe to hold it in position, about 20 cu. ft. of concrete being placed on each length

FIG. 10.— PRECAST CONCRETE BLOCKS.

of pipe. Sacks holding 2 cu. ft. of concrete were filled with a 1:2:4 mixture and tied. These sacks were then laid upon a section of pipe which had previously been buried in the ground to the mid-point of its diameter. By manipulation, these sacks of concrete were made to take the form of a semicircle hugging the pipe. Blocks made in this manner were laid on top of the pipe in the trench by a diver. They were placed near the joints first, and the process was continued until there were 14 blocks on each pipe. (See Fig. 10.)

Backfilling. While the pipe was being weighted by this means, the trench was backfilled with earth sufficiently to support the under side of

the pipe throughout its length, up to the mid-point of its diameter. After the weighting operation had been finished, the backfilling of the trench was completed, largely by the use of a drag bucket. In operating the bucket, care was taken not to pull it across the trench. In places where a scouring tidal action occurred, backfill consisted of sandbags placed by hand. Some of the backfill was placed by dumping it into the trench from a raft anchored in the river. After the work was completed by the Contractor, it was inspected by a diver employed by the Water Department.

Laying Pipe above Low-water Mark. From either end of that part of the pipe line which was laid below mean low water, the line was extended by laying Transite pipe, one length at a time, in a trench not less than 4 ft. deep. No concrete was placed on top of the pipe which was laid between high- and low-water marks, because it was believed that the backfill and the surrounding soil would be effective in holding the pipe in place. At either end of the Transite pipe line a connection was made with the bell-and-spigot cast-iron pipe of the distribution system. This was effected by means of a special cast-iron adapter, flanged to fit the Transite pipe on one end and equipped with a standard spigot on the other end. Backfilling was done by various methods, one of which was to sluice the excavated material into the trench by means of water from a fire hose.

Testing the Completed Line. After the entire installation of Transite pipe had been completed, it was thoroughly flushed and then tested. The first step in testing was to fill the pipe under the available normal pressure and to keep it full through a connection on which a meter and pressure gage were mounted. Then a pressure pump was applied and the water pressure raised to 100 lb. per sq. in., this pressure being maintained for two periods of 30 min. each. The total leakage in 506 ft. of flanged pipe, with 6 Mineralead joints in cast-iron pipe and one Ross regulator valve, was only 0.282 gal. in 30 min. This was less than 10 per cent. of the maximum leakage allowed by the specifications. Fire-flow tests, in which pumps and quick-closing nozzles were used, gave results that were satisfactory as regards flow of water, and failed to develop in the Transite pipe any weakness in ability to resist the resulting water hammer.

While the flow was at a maximum a fire-hose nozzle-gate was shut quickly, creating a severe water hammer, which not only broke a service pipe but cracked a length of 10-in. cast-iron pipe. The normal water pressure in the pipe is now about 50 lb. per sq. in.

On February 16, 1937 a portion of the Transite pipe buried in the marsh muck and gravel on the river shore was uncovered and was found to be in excellent condition.

Cost. Bids were received for laying the pipe for the river crossing, a total length of 500 ft., on the basis of (Item 1) 10-in. cast-iron pipe, part

bell and spigot pipe and part flexible joint pipe; and (Item 2 alternate) 10-in. Transite pressure pipe with flanged joints. The pipe in either case was to be furnished by the Water Department. Eleven bids were received and the contract was awarded to the low bidder. The average of 10 bidders for Item 1 was equivalent to $7.25 per lin. ft. and for Item 2 $7.20 per lin. ft. Six of the bids favored Transite pipe, three were the same for each type of pipe, and two favored cast-iron pipe. The Transite pipe cost $1.75 per lin. ft. including flanges, bolts and nuts.

The total cost of the river crossing was $6,135.

Acknowledgment. The Transite pipe river crossing was constructed under the direction of William J. Lumbert, Superintendent of the Scituate Water Department, with the advice of Metcalf & Eddy, Engineers. The work was performed by R. H. White, Contractor, Auburn, Mass. The Transite pipe was furnished by Johns-Manville Sales Corporation.

TRANSITE FOR WELLS AND SCREENS.

BY WM. J. LUMBERT.*

[*Read March 18, 1937*].

During the year 1931, the town of Scituate, Mass., purchased the property of the Scituate Water Co., which was then part of the Community Water Service holdings. The operation of the system was taken over by the town in July and has been carried on as a public utility since that time. The principal source of water supply was Old Oaken Bucket Pond which was inadequate for the peak loads in summer and due to the necessity of filtration carried an excessive overhead during the other months of the year. New sources of supply were mandatory at once, and work started by the Community Water Service, consisting of the construction of three deep wells, was pushed forward as rapidly as possible.

The geological formation in the vicinity of Scituate is such that it is apparently impossible to obtain a sufficient supply of water in any one locality. The three wells previously mentioned did not furnish a sufficient supply of water without the use of Old Oaken Bucket Pond, and as the ground water is much more desirable, continued prospecting has been carried on every year in the search for additional supplies.

Iron pipe in sizes from 2½ to 30 in. in diameter has been used in this search. Because iron pipe screens which had been in the ground only a few years were so badly corroded as to require replacement, a more suitable material was sought. After careful consideration of various materials, Transite pressure pipe was selected and is now being used for well screens and wellpipe. Transite pressure pipe is manufactured from a mixture of asbestos fibre and Portland cement. It is made up on revolving steel mandrels with a continuous thin sheet of the mixture applied under pressure. The resulting pipe has a dense, homogeneous wall, impervious and resistant to corrosion. Transite pipe is particularly suited for well pipe in gravel-packed wells and may also be used for the screen. The use of Transite will insure a non-corrosive material for the principal part of the well and will forever maintain a full flow of water and obviate the need and cost of removing corrosion.

In a gravel-packed well recently constructed in Scituate, Mass., a 16-in. Transite pipe was used. The pipe was standard Class C Transite, 16 in. inside diameter with a wall thickness of 1.44 in., each section being 13 ft. long. The weight was approximately 70 lb. per linear foot. Sliding-fit couplings 18 in. long, attached to the pipe with Parker Kalon screws, were used to join the lengths together. The total weight of the pipe and screen was approximately 4 600 lb., and using twelve screws on each end

*Superintendent, Water Department, Scituate Mass.

of the pipe, the maximum shearing force was slightly over 380 lb. on each. Parker Kalon screws of $\frac{3}{8}$-in. wire were placed in two rows, the rows being 6 in. apart at each end of the coupling, the screws being placed evenly and staggered around the pipe. This arrangement placed twelve screws in each end of each pipe, six in each row, and a total of twenty-four in each coupling. The screws did not penetrate the inside surface of the pipe.

Two sets of pipe clamps, made of wood, were used to place the pipe in the well; two 30-ft. poles with a crossbar formed the hoist; and a Ford

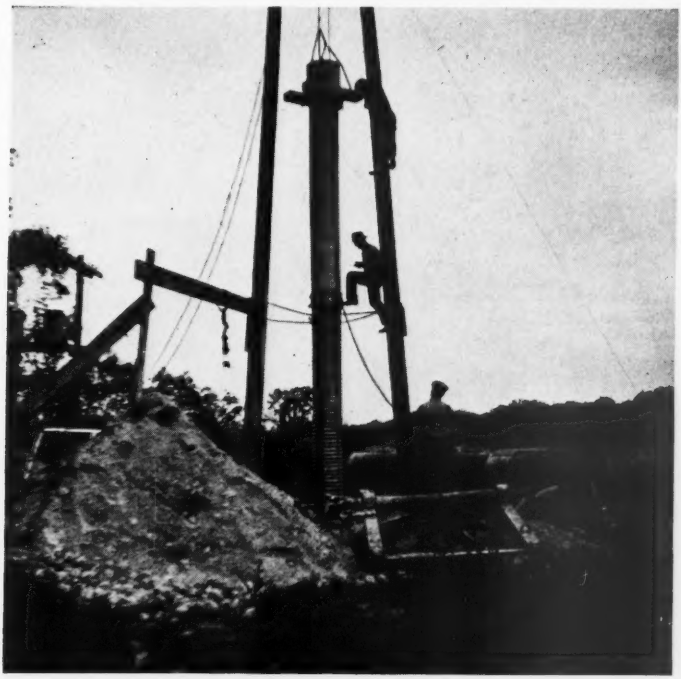


FIG. 1.— WELL SCREEN AND FIRST LENGTH OF 16-INCH TRANSITE PIPE READY TO BE LOWERED INTO WELL.

tractor was used for power. One set of clamps was placed on a piece of pipe near one end, and a coupling was set on the same end of the pipe. One row of screws was put in to hold the coupling firmly in place. A rope sling was placed over the ends of the clamps and the hoisting cable hook passed through the sling. The pipe was hoisted with the coupling up, swung over the well and lowered until the clamps rested on timbers laid across the top of the well. The coupling assembly was then completed on that pipe. The next piece of pipe was assembled in the same manner as the first, hoisted

over the well and lowered into the coupling on the pipe previously hung in the well. The pipe was carefully set in the coupling and forced home. The upper end of the coupling was then made fast to the upper length of pipe by boring holes with an electric drill and setting the Parker Kalon screws as designed. The pipe assembly in the well was then hoisted to swing free and the lower clamp removed. The pipe was then lowered till the next set of clamps rested on the timbers. This process was repeated

FIG. 2.— TRANSITE COUPLING READY TO RECEIVE SECOND LENGTH OF PIPE.

until the entire assembly was in the well. The well pipe was then lifted to swing free on the hoist, thus assuring the perpendicularity of the well. Washed and graded gravel from $\frac{1}{8}$ to $\frac{3}{8}$ in. in diameter was deposited around the well pipe up to the elevation of the bottom of a concrete seal which was placed later. The cable was slacked off, allowing the pipe to settle. The settlement was less than $\frac{1}{2}$ in. This completed the installation of a 16-in. Transite well pipe having a total depth of 60 ft., the entire operation being completed in less than two hours with only three men. (See Figures 1 to 3.)

FIG. 3.— DRILLING HOLES IN UPPER HALF OF TRANSITE COUPLING FOR PARKER-KALON SCREWS.

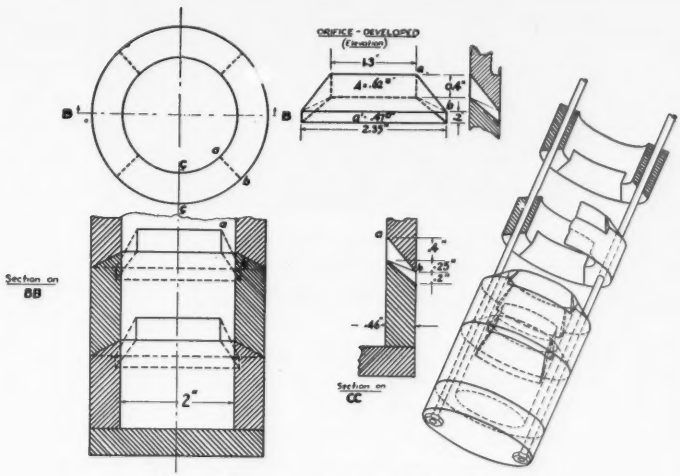


FIG. 4.— DETAIL OF A 2-INCH WELL SCREEN.

An interesting incident occured when the coupling that had been placed on the top of the last pipe was removed. This coupling was removed because the grade of the pumphouse floor had been lowered about 9 in. The screws were taken out, and two men attempted to lift the 100-lb. coupling, but it stuck fast because of the sliding fit. A small wire was then passed through two of the ⅜-in. holes at the top of the coupling and the

FIG. 5.— PORTION OF 6-INCH TRANSITE WELL SCREEN.
Showing rods at bottom used for attaching Louvre sections and bottom plate. Wide ring at top is used with Coupling for attaching to Transite Well Pipe.

hoist was hooked on. The coupling again refused to budge, but the entire assembly started to move. The 4 600 lb. were suspended on the small cable through the two holes, showing that the shear stress on the twelve screws was entirely safe. The pipe was allowed to settle (less than 1 in.) and the coupling was removed, without breakage, by the use of two 16-lb. hammers, driving up on the lower edge.

The construction of a Transite well screen may be accomplished in several ways. The sections are made up so that the top of one section fits securely into the bottom of the one next above, and all the sections are drilled with vertical holes so that they may be threaded on rods and properly held in place while being set in the well. The openings are made to fit the conditions found in the water-bearing strata, or the gravel which is placed around the screen, and also are governed somewhat by the amount of water to be drawn from the well. The opening is shaped so that the area on the outside of the pipe is longer and not as wide as the area on the inside of the pipe. This construction allows gravel that may be drawn through the narrow outside area to pass through the screen wall and drop to the bottom instead of being held in the opening and clogging the screen. (See Figures 4 and 5.)

In the smaller sizes it may be more feasible to bore holes through the shell of the pipe. These holes may be from ¼ to ⅜ in. in diameter, sloping upward toward the inside of the pipe. The sloping hole has an effect similar to the shutter type screen, allowing the sand to drop back on the outside of the pipe.

If the screen is on a suction line, it is necessary to use a coupling with the rubber ring rather than a slip-fit coupling, which is used in the well where the pump is set inside of the screen. Parker Kalon screws (or a similar arrangement) become necessary to stiffen the joint and also to take care of the tension while the screen and pipe are being lowered and set in the well. The screws must necessarily be placed near the ends of the coupling, beyond the rubber rings, in order to maintain a vacuum.

As in the shutter type of screen, the size of the opening would be regulated by the formation of the strata or the gravel used. In general the total area of the openings per linear foot of screen would be equal to one-third the area of the inside of the pipe as a minimum. The installation of Transite well pipe and screen offers no more difficulties than other types of screen or pipe, but the inherent structural conditions of Transite require thorough study of the methods of handling and placing. In many instances the work is actually simplified to a considerable extent. Transite can be used for well screens and pipe in sizes from 2 in. to 36 in.

For small wells, not over 75 ft. deep, the maximum vacuum that may be used on Transite with regular couplings should not be over 23 in. of mercury. This, however, is beyond the point of efficient operation of pumps in general and allows a reasonable factor of safety.

In a recent attempt to obtain an additional supply of water a hole was drilled and partially developed. A Transite screen and well pipe was set in the hole at a depth of 60 ft. and the annular space packed with ¾-in. gravel. The well was then surged with a piece of wrought-iron pipe wrapped with rope to fit closely inside the Transite, and pumped, alternately. The well did not develop the desired amount of water and it was decided to abandon operations at that point. There would have been a considerable

loss to leave the Transite screen in the hole, and a derrick, geared so that a very slow motion could be produced, was set up over the pipe. The cable was attached to the Transite and a very slow but steady pressure applied through the derrick. The pipe and screen held firmly and in less than one hour the entire 60 ft. of pipe and screen were pulled, taken apart and placed on the surface of the ground. The same screen and pipe was later set in a well about 100 ft. from the original location.

TRANSITE PIPE.

BY GERALD W. BLAKELEY.*

[*Received September 1, 1937.*]

Transite is an asbestos-cement product. A very strong, durable, non-metallic material is fabricated by combining fibre and cement under pressure. In this country Transite in flat and corrugated form has been manufactured since 1900. This material has been used extensively for many years where corrosion-resisting, fire-resisting and weatherproof construction is required. Besides its application as roofing and siding it has served to make flues, stacks, laboratory table tops, hoods and housings for removing industrial gases and moisture, for boiler and furnace casings and for applications needing unusual durability. Now this material, with all its advantages, is available in cylindrical form as a pipe for carrying water.

Asbestos-cement pipe under the trade name "Eternit" was first produced in Italy by Societa Anonima Eternit Pietra Artificiale of Genoa in 1913. Later the trade-name "Italit" was adopted for pipe exported from Italy. In each country where asbestos-cement pipe is manufactured and sold, an identifying trade-name is used, such as "Everite" in Great Britain and "Transite" in the United States and Canada.

This type of pipe has been manufactured and installed in various parts of the world for nearly a quarter of a century. In 1916 approximately 13 500 linear feet of asbestos-cement pipe were placed in service in Italy. Quantities installed each year grew rapidly until, in Italy alone, an average of nearly 800 miles per year was used after 1931. The manufacture in other countries started in 1927. Now twelve companies throughout the world have been licensed to use the machinery patented by the Societa Anonima Eternit Pietra Artificiale.

Manufacture. The dense, strong, durable nature of Transite pipe is due to the raw materials consisting of asbestos fibre and cement and the process by which these materials are intimately mixed and formed under pressure. The processed fibre which forms about 15 per cent. by weight of the composition, and the 85 per cent. of cement are intimately mixed and dispersed in water. A cylinder mold revolving in the resultant slurry transmits to an endless, porous felt a sheet about 1/100 in. in thickness. The felt carrying the sheet passes under a suction box which removes a large part of the water from the sheet and felt. The felt then passes over a press-roll and under the mandrel, on which the pipe is formed. By transference of the sheet to a polished, steel mandrel the pipe is formed. Pressure is applied to the material in process of formation by pressed-rolls

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which create a pressure of about 2 000 lb. per sq. in. Operation of the machine is continued until the pipe reaches the specified thickness, as indicated by a gage. (Fig. 1.)

The pipe is removed from the mandrel and placed on a pre-curing rack; a core is inserted at this point in order to insure roundness and the maintenance of specified dimensions during the pre-curing period. After adequate strength has developed on air-curing, the pipe is cured by a

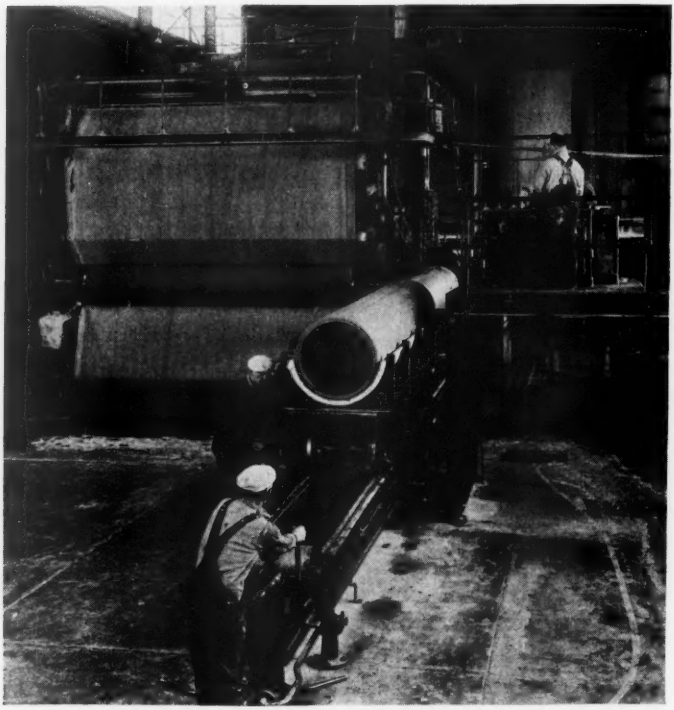

FIG. 1.— TRANSITE PIPE MACHINE.
Showing a Length of Pipe Being Removed from Mandrel.

special process to develop full strength and other properties which distinguish Transite Pressure Pipe from conventional Portland cement compositions. On completion of the curing process, the pipe is trimmed to length, and a second machine finishes the outside surface of both ends for use with the Simplex coupling.

In 1935, the manufacturing capacity of the Johns-Manville Corporation for producing Transite pipe was doubled at its Waukegan, Ill. factory. Early in 1936, more equipment was installed to triple the production, and

even then the plant could not keep up with the demand. Early in the fall of 1936, a new pipe plant was put in operation at Manville, N. J., with a capacity equal to that at Waukegan. This resulted in six times the original capacity, and the Manville plant, being so much nearer to New England points gave a source from which one-day delivery could be made by truck, if necessary.

FIG. 2.— TRANSITE PIPE BEING TESTED FOR FLEXURAL STRENGTH BY INSPECTION DEPARTMENT.

Inspection. An organization entirely separate from the manufacturing division has the responsibility for testing Transite pipe, couplings and rubber rings. Each length of pressure pipe and each coupling is hydrostatically tested to at least twice the working pressure. Flexural tests are conducted on the pipe in sizes up to 8 in. in diameter. The loads are applied at the third points, and the deflection is observed. Figure 2 shows a flexural testing machine. Each individual coupling and pipe length is stamped by the inspection department and Class 150 pipe in sizes from 4 to 24 in. bears the approval stamp of the Underwriters' Laboratories, Inc.

TABLE 1.—DIMENSIONS AND APPROXIMATE WEIGHTS OF TRANSITE PRESSURE PIPE.

Pipe Size, Inches.	Class 50.				Class 100.				Class 150.				Class 200.			
	Thickness. In.	Outside Dia. In.	Weight, Lb. Per Lin. Ft. Pipe Only.	Weight, Lb. Per Lin. Ft. Incl. Coup.	Thickness. In.	Outside Dia. In.	Weight, Lb. Per Lin. Ft. Pipe Only.	Weight, Lb. Per Lin. Ft. Incl. Coup.	Thickness. In.	Outside Dia. In.	Weight, Lb. Per Lin. Ft. Pipe Only.	Weight, Lb. Per Lin. Ft. Incl. Coup.	Thickness. In.	Outside Dia. In.	Weight, Lb. Per Lin. Ft. Pipe Only.	Weight, Lb. Per Lin. Ft. Incl. Coup.
2	.34	2.68	2.25	3.2	.41	2.82	2.8	4.0	.46	2.92	3.22	4.47	.50	3.0	3.56	4.88
2½	.34	3.18	2.75	3.86	.41	3.32	3.39	4.75	.46	3.42	3.87	5.27	.50	3.5	4.26	5.76
3	.36	3.72	3.43	4.7	.44	3.88	4.28	5.8	.49	3.98	4.86	6.43	.53	4.06	5.32	7.01
3½	.36	4.22	3.94	5.33	.44	4.38	4.92	6.58	.49	4.48	5.55	7.26	.53	4.56	6.06	7.93
4	.33	4.66	4.46	5.48	.48	4.96	6.53	7.95	.53	5.06	6.87	8.36	.60	5.20	7.7	9.32
4½	.34	5.18	5.15	5.73	.49	5.48	7.43	8.2	.55	5.60	7.96	8.77	.64	5.78	9.2	10.1
5	.35	5.70	5.85	6.5	.50	6.00	8.35	9.18	.57	6.14	9.1	9.97	.68	6.36	10.8	11.77
6	.36	6.72	7.15	7.9	.51	7.02	10.07	11.03	.62	7.24	11.8	12.8	.75	7.50	14.15	15.28
7	.38	7.76	8.76	9.87	.52	8.04	11.9	13.29	.66	8.32	14.45	15.96	.83	8.66	18.15	19.83
8	.42	8.84	11.05	12.03	.52	9.04	13.5	14.7	.69	9.38	17.12	18.28	.91	9.82	22.65	24.3
10	.44	10.88	14.40	15.6	.59	11.18	19.0	20.48	.85	11.70	26.6	28.4	1.10	12.20	34.1	36.25
12	.48	12.96	18.72	20.18	.68	13.36	26.1	27.86	.98	13.96	36.4	38.84	1.24	14.48	46.0	48.94
14	.52	15.04	23.6	25.6	.78	15.56	35.0	37.72	1.13	16.26	48.8	52.54	1.44	16.88	62.0	66.48
16	.56	17.12	29.0	31.36	.88	17.76	45.0	48.39	1.25	18.50	61.6	66.26	1.65	19.30	81.5	87.22
18	.59	19.18	34.2	36.88	.97	19.94	56.1	60.3	1.39	20.78	77.0	82.78	1.87	21.74	104.0	111.13
20	.63	21.26	40.5	43.55	1.07	22.14	68.4	73.5	1.53	23.06	94.4	101.4	2.09	24.18	128.0	137.1
24	.69	25.38	53.2	57.8	1.25	26.50	95.6	103.62	1.82	27.64	134.5	145.7	2.48	28.96	183.0	196.5
30	.90	31.80	86.6	93.9	1.54	33.08	147.5	161.7	2.29	34.58	210.0	230.5	3.12	36.24	288.0	310.6
36	1.09	38.18	126.0	136.35	1.83	39.66	211.0	231.1	2.80	41.60	310.0	338.5	3.74	43.48	415.0	447.5

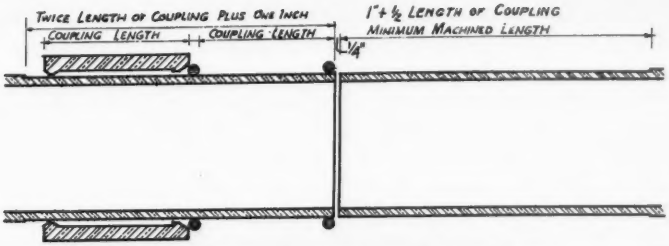

FIG. 3.— SIMPLEX COUPLING AND RUBBER RINGS.
In Starting Position for Joining Transite Pipe.

FIG. 4.— VIEW OF TRANSITE PIPE.
With Simplex Coupling and Rubber Rings Ready to Make Joint.

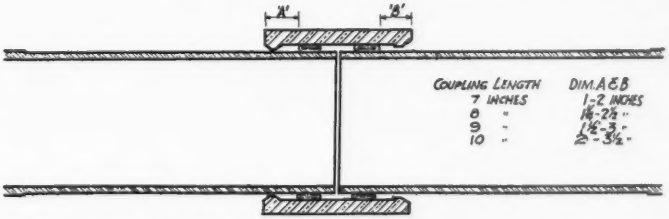

FIG. 5.— SIMPLEX COUPLING ASSEMBLED ON TRANSITE PIPE.

Description. Transite pipe is made in sizes from 2 to 36 in. inclusive. Working pressures in pounds per square inch and equivalent heads in feet of water are shown below:

Class.	Working Pressure—lb. per sq. in.	Equivalent Head—ft. of water.
50	50	115
100	100	231
150	150	346
200	200	462

Standard lengths are 13 ft. for sizes down to 4 in. In sizes smaller than 4 in., 5 ft. lengths are furnished in order to provide flexible couplings at more frequent intervals. The pipe is manufactured slightly oversize to allow the plain ends to be machined to the outside diameters shown in Table 1. Both ends of each length are machined for a distance sufficient to accommodate the couplings used for joining the pipe.

Simplex Couplings. Simplex couplings are usually employed for joining Transite Pressure Pipe. Figures 3 and 4 show the Simplex coupling before assembly and Figure 5 shows it after the joint has been made. The coupling is a sleeve of Transite, of proper thickness to give the necessary strength, machined accurately on its inside diameter. The inside diameter of the coupling provides just a sufficient space outside of the machined end of the pipe to squeeze the rubber rings to one-half their original thickness throughout their entire circumference. On one end of the coupling, an internal taper is provided to allow the rubber rings to enter the annular space between the pipe and coupling when the latter is drawn lengthwise along the pipe toward the joint. Figure 6 shows the hydraulic coupling puller employed. At either end of the coupling the machining provides a shoulder projecting inward. The forward shoulder, where the rubber rings enter, projects inward just sufficiently to allow the operator of the coupling puller to feel the difference of pull required as the ring passes into the coupling. The shoulder on the back end of the coupling is made sufficiently high to act as a stop to the movement of the coupling if the operator pulls it too far.

Rubber Rings. Moulded rubber rings of very high quality are used for making Simplex couplings watertight. The initial cross-sectional diameter of these rings is 0.55 in. for pipe from 4 to 12 in. in size. For smaller sizes of pipe, rings have an original diameter or thickness of 0.47 in., but for larger sizes they increase in thickness until a diameter of 1.00 in. is used for 36-in. pipe.

The rings contain a minimum of 75 per cent. of pure Para rubber. No scrap rubber or rubber substitute is used in their compounding. Anti-oxydents and aging compounds are added to produce maximum resilence,

the greatest tensile strength and the lowest cold-flow properties. This type of rubber should not be confused with that used in elastic bands, overshoes and pencil erasers. In the majority of these materials such a large proportion of filler is introduced that the true characteristics of rubber are lost.

There is a great deal of evidence to prove the lasting properties of rubber to keep pipe joints tight, particularly when the rubber is buried

FIG. 6.— HYDRAULIC COUPLING PULLER.

under ground in the presence of water and dampness, with the deteriorating influences of light, heat and air eliminated. A few of the cases showing the permanence of rubber for pipe joints are given below:

1. Rubber as a pipe-jointing material dates back to the nineteenth century; it was used on a pipe line in England in 1863 and in France, at Nice and Lyons, in 1870; in the United States its use goes back approximately as far as in European installations.

2. In 1903 Walter Hole laid a line 450 ft. long at Torquay, England with rubber joints because of trouble experienced with settling and shifting earth. He pronounced the life of the rubber used as unlimited.

3. In 1870 in Elizabeth, New Jersey, rubber was used for joints in a pipe line. In 1931, 61 years later, when the line was removed, the rubber jointing material was found to be in a fine state of preservation.

4. The Eternit Company, manufacturers of Asbestos-cement pipe in Italy have been using rubber rings in the Simplex-type of joint for over 20 years; in all their experience with this type of joint in water service, they have never encountered an instance where the rubber ring has disintergrated or failed.

Fittings and Valves for Use with Transite Pipe. With Transite pipe, cast-iron fittings and valves of the all-bell type are used, and joints are made according to general practice for bell and spigot type joints. Figure 7 shows a cast-iron fitting in a Transite line. Fittings may be lined if the owner or his engineer so desires. Class D fittings with standard-sized bells may be used with Transite pipe except as noted below:

Class.	Fittings with oversized bells are required for Transite pipe of the following sizes.
50	Not required
100	Not required
150	10 in. or larger
200	6 in. or larger

Oversized bells on valves are only required as noted for fittings.

Tapping Transite Pipe. Transite pipe is tapped in accordance with regular waterworks practice and by the use of standard tapping tools. It is advisable to tap about ⅛ in. deeper to allow the threads of the corporation stop to extend entirely through the material when Class 150 Transite

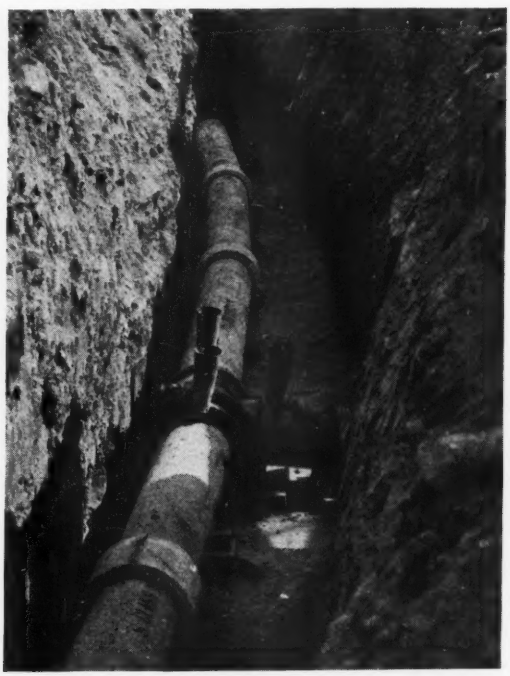

FIG. 7.— POURED JOINTS IN CAST-IRON FITTING.

pipe is used. As recommended by the manufacturers of corporation stops and tapping tools, the tap should have the same taper and the same number of threads as the corporation stop to be used. The tap ought to be sharp so that it will be able to cut the strong asbestos reinforcing fibre in the pipe. It is advisable to use cutting oil on the tap and graphite and grease on the corporation stop.

Characteristics of Transite Pipe. In the first place Transite is one of the most enduring materials for use under ground. The smooth interior imparted to it by the polished, steel mandrel on which it is made gives it a very high carrying capacity with unusually low frictional resistance to the flow of water. This results in a Williams and Hazen coefficient C of 140 or better. This high flow characteristic is continuously maintained as the material is unaffected by the agencies which so often roughen the inside lining of water pipes composed of other materials. The use of Transite pipe results in higher fire-flow when it is needed.

Transite pipe is light in weight and can be easily handled. The ease of handling and the simple and quick method of jointing the material has led many contractors, after they have once laid Transite pipe to make their estimates lower for laying it than for other kinds of pipe of equal sizes. The immunity of Transite pipe to ill effects from corrosive water, high in carbon dioxide, of which we have so much in New England, has been one of the reasons for its being recommended in many places.

Underwriters' Laboratories Recommendations. On February 26, 1937 the Underwriters' Laboratories, Inc., maintained and operated by the National Board of Fire Underwriters, granted their recommendation No. Ex. 1251 of January 20, 1937 for the use of Transite pipe, Class 150, in public and private water works and fire-service systems. This approval also covered the use of Simplex couplings, of bell-end cast-iron fittings and valves with joints made of self-caulking, pouring compounds and of the tapping of Transite pipe by standard methods. While Transite pipe is manufactured in sizes up to and including 36 in., the Underwriters have listed, as with other types of pipe, sizes up to 24 in. only. Transite pipe may, therefore, be included in specifications by calling for pipe for water mains as recommended by the Underwriters' Laboratories, Incorporated.

NEW ENGLAND DROUGHTS AND FLOODS IN THEIR RELATIONSHIP TO WATER SUPPLY.

BY CALEB MILLS SAVILLE.*

[*Read September 23, 1936.*]

"Men are wise in proportion, not to their experience, but in their capacity for experience."†

The rather remarkable antithesis of excess and deficiency in precipitation which has been experienced in Southern New England during the past decade and a half has been of particular interest to Metropolitan Hartford, (Conn., U. S. A.) on account of its bearing on local water supply problems. To a populous district dependent upon surface water stored in reservoirs, the vicissitudes of precipitation and stream flow are of vital moment. Excess of rainfall‡ in limited periods produces floods for which provision must be made to prevent damage to life and property. Deficiency in rainfall, if long continued, produces drought, which must be anticipated so far as possible in order that the supply may be maintained.

While experience is acknowledged to be an excellent teacher, she is also known at times to lay a stiff price for her trouble. Excess of caution on the one hand may lead to unwarranted expenditure of funds, while lack of knowledge (experience) may result in hardship and irreparable loss. The facts presented here are intended only to add to the store of recorded experiences, and little or no attempt is made to present special theories provocative of argument.

The Hartford Water Supply Works have rainfall records extending back for three-quarters of a century and streamflow records for the last third of that period (Nepaug and East Branch 1912–1936). For rainfall the maximum calendar year has been that of 1920, with 56.95 in., and the minimum that of 1935, with 28.90 in. (West Hartford Station 1868–1936). Pertinent to the relationship between these two extremes is the remark of Rafter§ that, "as a general rule the minimum rainfall may be placed at about one-half the maximum" and that of Symons** that, "a rule used to be quoted and it was approximately, but not absolutely right, that the rainfall in the wettest year would be twice what it was in the dryest year." These remarks of experienced observers would indicate that the past 100 years for this locality has compassed the range in rainfall reasonably to be anticipated. The distribution of these amounts in individual years throughout the period, however, may bring about such entirely different conditions

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†George Bernard Shaw "Maxims for Revolutionists."
‡In this paper "rainfall" is used in the generic sense.
§U. S. Geol. Survey, W. S. & I. paper No. 80, p. 12.
**Proc. Inst. Civ. Eng. *109* (1892), p. 58.

during years of essentially equal rainfall that it is impossible to generalize too closely as to effects even on drainage basins in the same locality.

In October, 1924, the author prepared a paper* for this ASSOCIATION detailing some experiences with flood and drought on the Hartford areas and this matter was somewhat further discussed in later papers.† Additional knowledge and more recent experiences with these phenomena have accumulated since then, and it is the purpose of this paper to add to the record.

Although drought experiences are ordinarily less spectacular than those of flood, it often happens that the results of the former cause more distress and ultimate loss than the latter. It is proposed, therefore, to consider, first the matter of drought, following with floods, which is the order of these happenings due to combinations of meteorological phenomena recently experienced in this part of the country. In the past year weather seems to have run the gamut from the low water of 1935 to the floods of early 1936, although the drought period actually began with the year 1930 and continued with more or less severity until 1936.

Before proceeding with the special subjects, it seems proper to make some general observations regarding the causes that produce those conditions of drought and flood which the ordinary person often considers abnormal.

Generally speaking, the meteorological phenomena that we designate as "weather" are the result of combinations of natural forces, mostly due, however, to differences in the amount of heat received from the sun and dependent on the position of the surface of the earth exposed to solar rays. The variations that most appeal to us are departures from the normal course as experienced over a long period.

Drought, in its usual meaning, indicates lack of rain for appreciable periods, and the variable conditions influencing rainfall are dependent mostly upon changes in barometric pressure, temperature and wind movement, modified by configuration of the land, its exposure to prevailing winds and its proximity to large bodies of water. As a general principle, a rainfall is due to sudden cooling of the air below the "dew point," whereby the water vapor always present in the air is condensed to a liquid state and floats away in form of clouds or is precipitated in form of rain.

Many theories have been proposed as to the reasons and reactions necessary to produce this cooling. All gaseous matter, and air is included in this classification, is cooled by expansion. If air is forced upward by some means into the cooler spaces of lower pressure, it expands and drops its burden of water vapor; as for example, when a hot moist current of air meets rising ground, or when a wedge of cold air is pushed beneath a stratum of warm moist air forcing it upward.

There are three principal types of rainfall; first, by an air current forced up into colder localities as in meeting ascending mountain slopes (oro-

*This JOURNAL, *39* (1925).
†An. Rep. Conn. Soc. C. E. 1935, p. 97, and this JOURNAL, *47*, p. 363.

graphic), or coming in contact with colder ground; second, by the unequal heating of the lower layers in contact with the ground, causing unstable conditions and a rush of air upward (convection) similar to conditions in a heated room where, although the source of the heat is at the floor level, the highest temperature is at the ceiling; the third method and that most productive of floods and droughts is found in the general circulation of

LEGEND
Full Line —— Storm of March 6-12, 1936.
Broken — — — — 17-23, "
(B)—Location of Low Pressure Areas.
(H)— - - - High - - -
Number in Circle representing day of Month.
Roman Numerals indicate separate storm tracks.
Data taken from daily Weather Maps compiled by the U.S. Weather Bureau.

FIG. 1.— TRACKS OF STORMS CAUSING MARCH, 1936, FLOOD.

the atmosphere with its so-called cyclonic (lows) and anticyclonic (highs) phenomena found in the unceasing march of barometric depressions from west to east, particularly in the temperate zone.

The ordinary path of these depressions is somewhat widely defined, but in general they pass to the west and north of New England, and out through the St. Lawrence Valley, or up along the Atlantic coast and out to sea before reaching New England. A good example of the diverse paths

of these storms appears in the conditions which produced the flood of March, 1936, and shown on Fig. 1., by the deflection west and north in the full and dotted lines of the storm tracks shown on the diagram, particularly Numbers I, II and III, as these storms ranged up the Atlantic Coast. These storm tracks were assembled from the daily weather charts issued by the Weather Bureau. A copy of the resulting rainfall map (Fig. 2), shown on p. 4 "Climatological Data," U. S. Weather Bureau for March 1936, is included for purpose of information only as to the extent and variations of this storm. This sketch appeared at so early a date after the

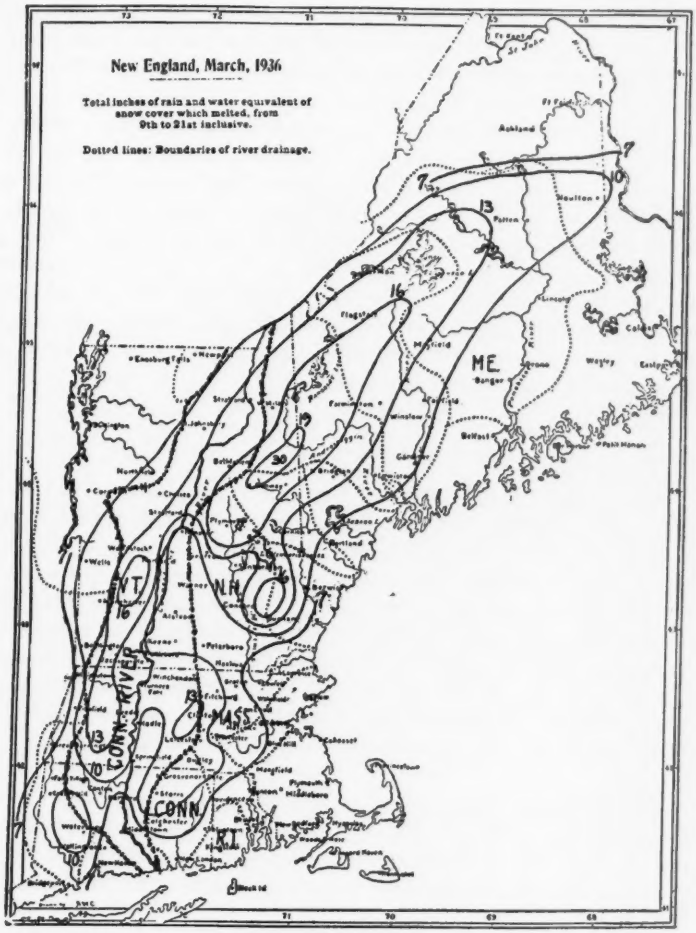

FIG. 2.— TOTAL RAINFALL AND WATER EQUIVALENT OF MELTED SNOW INVOLVED IN MARCH, 1936, FLOOD.

storm, in order to give the public some information, that it seems very likely some additions and corrections will be necessary.

When, for some reason more or less obscure, depressions from the west or south are diverted considerably from their courses and forced toward New England, storms are experienced and floods may follow — if turned away from New England, fair weather follows, which if too long continued results in drought: "The wind bloweth where it list'th, and thou hearest the sound thereof, but can'st not tell whence it cometh or whither it goeth."*

The late A. J. Henry† once remarked that, "it is quite probable that rain falls in the United States at one point or another every day of the year," and also, "while some sections of the country are parched from drought, others as well may be drenched by excess of rain." In the month of July, 1936,‡ for example, the weather map of the Continental United States shows only about 25 per cent. of the normal rainfall received in Southern New England, against 200 per cent. in the south, while for March, 1936,§ Southern New England had an excess amounting to some 200 per cent. and a region in Southern Kansas and much of Oklahoma had little in relation to normal precipitation. For the four months, January to April inclusive,** precipitation was 150 per cent. in New England and the west, with only 15 to 50 per cent. in some states in the lower Mississippi Valley. As a general condition, however, the middle west recently has suffered from prolonged deficiency in rainfall, while the northeast part has had sufficient, even if at times rather slack.

Drought.

New England is classified as lying in a humid region of the United States — nevertheless, there are times when those responsible for water supply are at their wit's end, as it were, to keep their communities supplied, because, "when the devil was ill the devil a saint would be, but when the devil got well a devil a saint was he"; that is, because New England as a normal condition is well supplied, it often is hard to coax money from reluctant government agencies to be spent on works for future protection, when the present seems so secure with water running to waste over the spillway. "Sufficient unto the day is the evil thereof," often seems a favorite rule for municipal governments, but the evil often comes "like a thief in the night."

Drought and wet periods come and go in an almost regular procession, but usually these are not experienced over the whole country at the same time, so regardless of the country as a whole, local conditions of rainfall and stream flow must govern the steps to be taken for meeting individual

*St. John III, 8.
†Chief of Div. U. S. Wea. Bu., Annual Rep. 1896–97, p. 317.
‡*Weekly Weather and Crop Bulletin*, U. S. Dept. Agr., August 4, p. 4.
§*Idem*, April 7, p. 4.
**Idem, May 5, p. 4.

situations. The period 1892 to 1896 and the present 1936, are some of the great drought periods in many parts of the country, but are not of particular moment to Hartford's water supply system. While the period 1879 to 1883 was Hartford's dryest period, the rainfall at New Haven, 30 miles to the south, was well above normal.

What may be the most lasting impressions of drought depends in great part on "whose ox is gored." To the farmer, lack of rain at the growing period means parched soil with crops destroyed or badly damaged; to the water supply man, lack of rain, at what "Rafter" has called the replenishing period (September to October inclusive), is of most account, and stormy weather delayed until then makes the farmer happy.

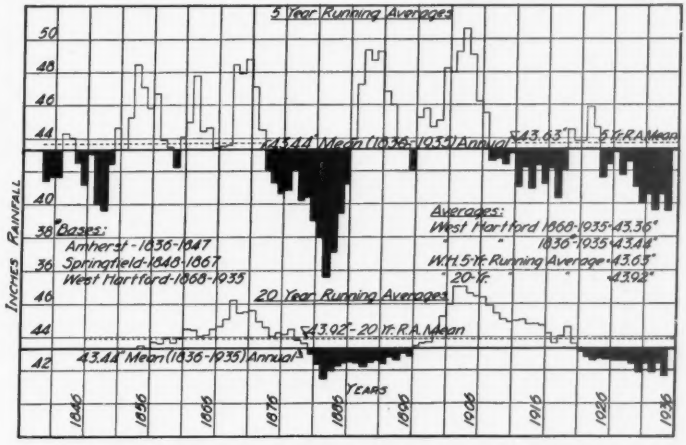

FIG. 3.— WEST HARTFORD, CONN., RAINFALL, 5- AND 20-YEAR RUNNING AVERAGES.

There are many other matters also beside meager rainfall that help or hinder the yield of streams for water supply purposes. Among these as stated are the distribution of the rainfall throughout the year, also the topography and geological character of the gathering grounds, their exposure to the rain-bearing winds and the vegetative cover of the drainage area.

Principally, however, in order to make reliable estimate of watershed yield, — and this paper is primarily concerned with that phase of drought, — it is necessary to have a reliable idea of rainfall conditions extending over a long term of years. It has been shown* that to cover departures from normal rainfall within limits of 1 per cent., a record of at least 50 years is desirable. Even the longest human records of rainfall, however, are as mere patches on actual weather history, as said the Psalmist, "a thousand years in thy sight are as yesterday when it is passed and as a

*Saville, This JOURNAL, 47, p. 414.

watch in the night,"* nevertheless, what has happened in the past, may be expected in the future, and that future may have in store far greater extremes than have as yet been recorded.

The Hartford water supply rainfall records (West Hartford) date from 1868, but in order to get a better picture of rainfall conditions here, a table by months has been computed back to 1836, using relationships to the longer period records of Amherst and Springfield, Massachusetts.

The results of this computation are presented in Fig. 3 in 5- and 20-year running averages for this 100-year period, referred to the average of 43.44 in. It is to be noted that the means of the 5- and 20-year averages are somewhat greater than those of the years taken singly. From the 5-year diagram it appears that there have been four periods of extremely low rainfall at this station; namely, centering about 1846, 1881, 1911 and 1931, the lowest 5-year averages being as follows:

For the 5 Years Beginning.	Average Rainfall, Inches.	Per cent. of 43.63 in. Long-period Average.	Lowest Year in the Period.	Annual Amount, Inches.	Per cent. of Average.
1845	39.70	91.0	1849	34.74	79.6
1879	35.72	81.9	1883	33.64	77.1
1910	41.01	94.0	1914	35.54	81.5
1931	39.66	90.9	1935	28.90	66.2

From this tabulation, it appears that the period beginning in 1879 was the lowest of record, and as far as now appears that, it extended over the longest period of consecutive years; the rainfall being below the average from 1874 to 1886, a period of 15 years, with an average of 40.40 in., which is 3.23 in., or about 7.4 per cent. below the long-period average.

For the period of record, the march of rainfall shows a wave-like progress when plotted by departures from averages. This tendency is shown graphically on Fig. 4, recording the course of rainfall at West Hartford station for 100 years.

It is to be noted that the base used in this diagram is 0.11 in. higher than that of the 5-year Diagram in Fig. 3. This difference is due in part to the inclusion of the 1935 record in Fig. 3 and its omission in Fig. 4, which has not been brought to-date. Note that this one-year record of 1935 reduced the 100-year average by 0.11. in.

As indicating the chance of rainfall at this station, Fig. 5 gives observations for one- and five-year periods. Although the low amount (28.90 in. in 1935) was not available when the analysis was made, it seems improbable that the trends of the curves will be altered to any appreciable extent.

It is by the rainfall of shorter periods than a year (12 months) that water supply may be most seriously affected. For the period of record at the West Hartford station, Tables 1 and 2 are presented for consecutive months and calendar years. From study of these data, it appears that

*90th Psalm, 4th verse.

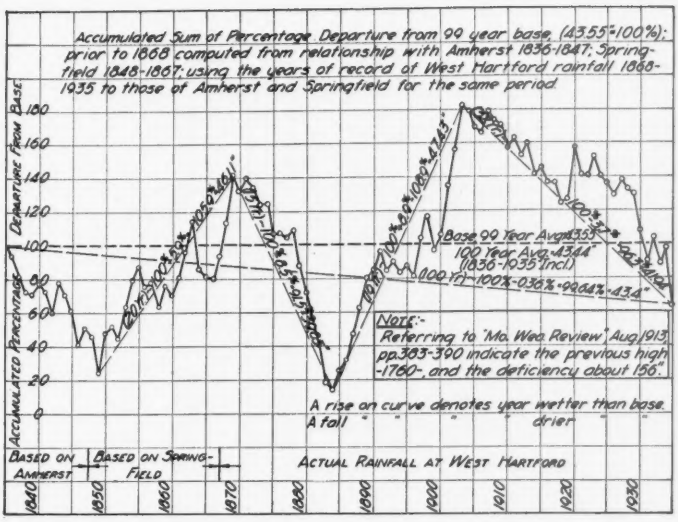
Accumulated Sum of Percentage Departure from 99 year base (43.55"=100%): prior to 1868 computed from relationship with Amherst 1836-1847; Springfield 1848-1867; using the years of record of West Hartford rainfall 1868-1935 to those of Amherst and Springfield for the same period.
Departure from Base
Accumulated Percentage
Base 99 Year Avge. 43.55"
100 Year Avge. 43.44" (1836-1935 incl.)
100 Yr. 100% - 0.36% = 99.64% = 43.4"
Note:- Referring to "Mo. Wea. Review" Aug. 1913, pp. 383-390 indicate the previous high -1760-, and the deficiency about 156".
A rise on curve denotes year wetter than base.
A fall " " " " drier " "
Based on Amherst
Based on Springfield
Actual Rainfall at West Hartford

FIG. 4.— WEST HARTFORD, CONN., RAINFALL, ACCUMULATED PERCENTAGE DEPARTURE FROM NORMAL.

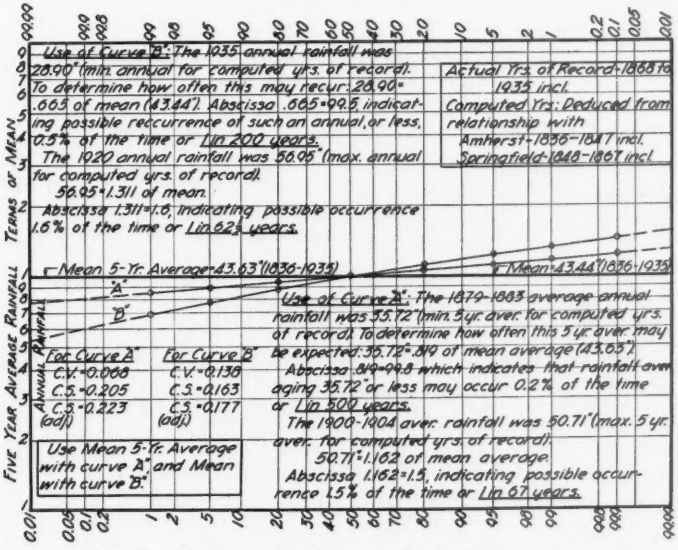
Use of Curve "B": The 1935 annual rainfall was 28.90" (min. annual for computed yrs. of record). To determine how often this may occur: 28.90" = .663 of mean (43.44"). Abscissa .665 = 99.5 indicating possible occurrence of such an annual, or less, 0.5% of the time or 1/n 200 years.
The 1920 annual rainfall was 56.95" (max. annual for computed yrs. of record). 56.95" = 1.311 of mean.
Abscissa 1.311 = 1.6, indicating possible occurrence 1.6% of the time or 1/n 62½ years.
Actual Yrs. of Record: 1868 to 1935 incl.
Computed Yrs: Deduced from relationship with Amherst - 1836-1847 incl. Springfield - 1848-1867 incl.
Mean 5-Yr. Average 43.63" (1836-1935)
Mean 43.44" (1836-1935)
"A"
"B"
Use of Curve "A": The 1879-1883 average annual rainfall was 35.72" (min. 5 yr. aver. for computed yrs. of record). To determine how often this 5 yr. aver. may be expected: 35.72" = .819 of mean average (43.63").
Abscissa .819 = 99.8 which indicates that rainfall averaging 35.72" or less may occur 0.2% of the time or 1/n 500 years.
The 1900-1904 aver. rainfall was 50.71" (max. 5 yr. aver. for computed yrs. of record). 50.71" = 1.162 of mean average.
Abscissa 1.162 = 1.5, indicating possible occurrence 1.5% of the time or 1/n 67 years.
For Curve "A": C.V. = 0.068, C.S. = 0.205, C.S. = 0.223 (adj.)
For Curve "B": C.V. = 0.139, C.S. = 0.163, C.S. = 0.177 (adj.)
Use Mean 5-Yr. Average with curve "A" and Mean with curve "B".
Annual Rainfall
Five Year Average Rainfall
Terms of Mean

FIG. 5.— PROBABILITY CURVES OF ANNUAL RAINFALL AT WEST HARTFORD, CONN.

TABLE 1.—MINIMUM RAINFALL AT RESERVOIR NO. 1, WEST HARTFORD, FOR CONSECUTIVE MONTHS AND CALENDAR YEARS.

No. of Consec. Months.	Date of Period.	Rainfall for Period.		Ave. Monthly Rainfall, Years of Record, Inches.	Per cent. Period (Col. 2) of Years of Record, (Col. 2 ÷ 3).	Per cent., Col. 1 of Average Annual Rainfall.
		Total, Inches.	Average Monthly, Inches.			
		1	*2*	*3*	*4*	*5*
1	Jun. 1873.....	0.15	0.15	3.32	4.5	0.3
2	Jan.–Feb. 1849.....	1.16	0.58	3.23	18.0	2.7
3	Jul.–Sep. 1870.....	3.70	1.23	3.98	30.9	8.5
4	Dec. 1838–Mar. 1839	6.03	1.51	3.36	44.9	13.9
5	May 1870–Sep. 1870	9.03	1.81	3.82	47.4	20.8
6	Sep. 1837–Feb. 1838	11.59	1.93	3.47	55.6	26.7
7	Sep. 1837–Mar. 1838	13.28	1.90	3.48	54.6	30.6
8	Sep. 1837–Apr. 1838	15.41	1.93	3.47	55.6	35.5
9	Aug. 1837–Apr. 1838	18.14	2.02	3.58	56.4	41.8
10	Sep. 1879–Jun. 1880	20.55	2.06	3.49	59.0	47.3
11	Sep. 1874–Jul. 1875	25.47	2.32	3.54	65.5	58.6
12	Aug. 1837–Jul. 1838	28.38	2.37	3.62	65.5	65.3
15	Oct. 1934–Dec. 1935	37.13	2.48	3.63	68.3	85.5
18	Jan. 1879–Jun. 1880	47.54	2.64	3.55	74.4	109.4
24	Oct. 1879–Sep. 1881	68.49	2.85	3.62	78.7	157.7
30	Jun. 1929–Nov. 1931	85.89	2.86	3.66	78.1	197.7
36	Sep. 1879–Aug. 1882	101.95	2.83	3.62	78.2	234.7
48	Sep. 1879–Aug. 1883	138.83	2.89	3.62	79.8	319.6
60	Jan. 1879–Dec. 1883	178.58	2.98	3.62	8.23	411.1
120	Oct. 1874–Sep. 1884	380.60	3.17	3.62	87.6	876.2

50 Year Normal (1878–1927) = 43.84 in. Average Annual Rainfall (1836–1935) = 43.44 in.

TABLE 2.—MINIMUM RAINFALL AT RESERVOIR NO. 1, WEST HARTFORD, FOR CONSECUTIVE PERIOD YEARS.

No. of Consecutive Months.	Date of Period.	Average 12 Months Rainfall, Inches.	Per cent. of Average Annual Rainfall.
12	Aug. 1837–Jul. 1838	28.38	65
24	Oct. 1879–Sep. 1881	34.25	79
36	Sep. 1879–Aug. 1882	33.98	78
48	Sep. 1879–Aug. 1883	34.71	80
60	Jan. 1879–Dec. 1883	35.72	82
120	Oct. 1874–Sep. 1884	38.06	88

No. of Years.	Years.	Average Yearly Rainfall, Inches.	Per cent. of Average Annual Rainfall.
1	1935.....	28.90	67
2	1930–1931.....	34.94	80
3	1881–1883.....	35.86	83
4	1880–1883.....	36.12	83
5	1879–1883.....	35.72	82
10	1875–1884.....	38.74	89

50 Year Normal (1878–1927) = 43.84 in. Average Annual Rainfall (1836–1935) = 43.44 in.

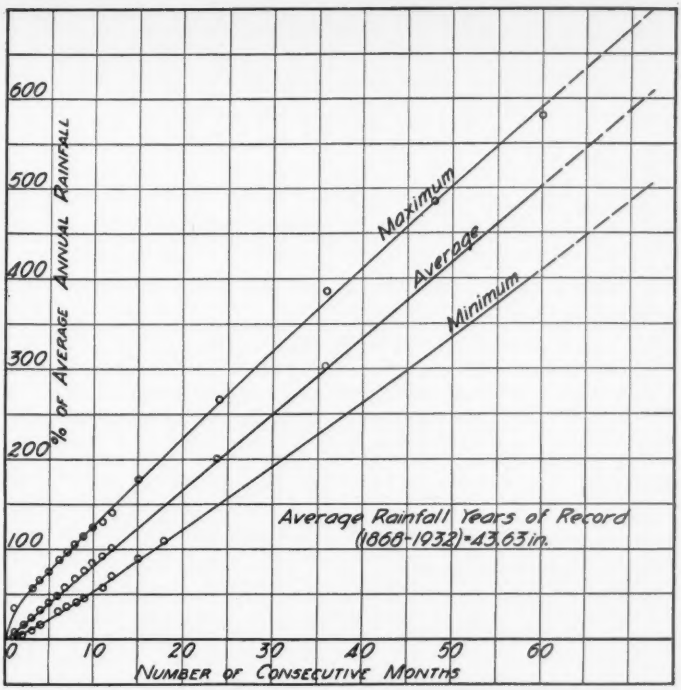

FIG. 6.— RATIO OF CONSECUTIVE MONTHLY RAINFALL AT WEST HARTFORD, CONN., TO AVERAGE ANNUAL RAINFALL.

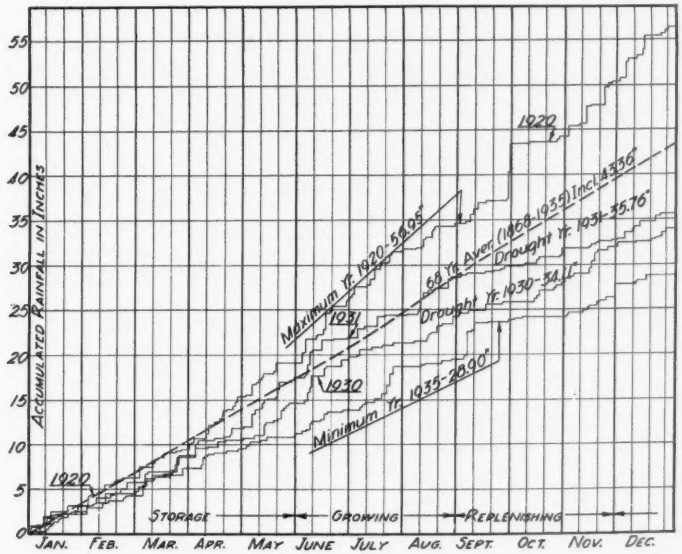

FIG. 7.— ACCUMULATED DAILY RAINFALL AT WEST HARTFORD, CONN.

TABLE 3.—MINIMUM RUN-OFF FOR CONSECUTIVE MONTHS AND CORRESPONDING RAINFALL, NEPAUG RIVER WATERSHED.

No. of Consec. Months.	Date of Period.	Run-off for Period.			Rainfall for Period.			Per cent. Period Rainfall to Rainfall (2 ÷ 3).	Per cent., Col. 1, of Average Annual Run-off.	Per cent. Collection, (1 ÷ 4).
		Total.	Inches. Per Month.	Ave. Monthly Years of Record.	Total.	Inches. Per Month.	Ave. Monthly Years of Record.			
		1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
1	Sep. 1931.....	0.123	0.123	0.660	1.01	1.01	3.620	18.6	0.5	12.2
2	Sept. –Oct. 1931.....	0.261	0.131	0.905	3.20	1.60	3.710	14.5	1.2	8.2
3	Aug. –Oct. 1930....	0.455	0.152	0.873	7.42	2.47	3.873	17.4	2.0	6.1
4	Aug. –Nov. 1931....	0.762	0.191	1.090	7.93	1.98	3.865	17.5	3.4	9.6
5	Jul. –Nov. 1931....	1.218	0.244	1.030	10.56	2.11	3.828	23.7	5.4	11.5
6	July –Dec. 1931....	1.813	3.302	1.147	13.43	2.24	3.733	26.3	7.7	13.5
7	July 1930–Jan. 1931	2.343	0.335	1.267	19.60	2.80	3.681	26.4	10.4	12.0
8	Jul. 1930–Feb. 1931	2.892	0.361	1.337	21.38	2.67	3.594	27.0	12.8	13.5
9	Jun. 1930–Feb. 1931	3.746	0.416	1.332	25.30	2.81	3.588	31.2	16.6	14.8
10	May 1930–Feb. 1931	4.952	0.495	1.456	29.28	2.93	3.633	34.0	22.0	13.5
11	Apr. 1930–Feb. 1931	6.894	0.627	1.667	31.45	2.86	3.338	37.6	3.06	21.6
12	Mar. 1930–Feb. 1931	9.434	0.786	1.877	35.23	2.94	3.627	41.9	41.9	26.8
15	Aug. 1929–Oct. 1930	13.852	7.923	1.676	46.01	3.07	3.676	55.1	61.5	30.1
18	Jul. 1929–Dec. 1930	15.190	0.844	1.634	53.20	2.96	3.662	51.7	67.4	28.6
24	Apr. 1930–Mar. 1932	25.990	1.083	1.877	75.61	3.15	3.627	57.7	115.4	34.4
30	Jul. 1929–Dec. 1931	30.626	1.021	1.731	90.03	3.00	3.648	59.0	135.9	34.0
36	Jul. 1929–Jun. 1932	40.556	1.127	1.877	110.15	3.06	3.627	60.0	180.0	36.8
48	Oct. 1928–Sep. 1932	61.670	1.285	1.877	154.24	3.21	3.627	68.5	273.7	40.0
60	Jan. 1928–Dec. 1932	93.358	1.556	1.877	207.30	3.46	3.627	82.9	414.4	45.0

Average Monthly Rainfall for Years of Record (1912–1932) = 43.52 in.
Average Monthly Run-off for Years of Record (1912–1932) = 22.53 in.

for the whole record extending back to 1836, no lesser amounts were found, than appear during the actual record since 1868, indicating that for about 100 years at least the records of the period from 1881 to 1884 were the minima for the century, except for the one year of 1935.

The distribution of rainfall throughout the year, as has been said, is of the greatest importance in water supply work, and Fig. 6 has been prepared to show maximum and minimum conditions throughout the years and their relation to the average for the years of record. A similar diagram and the prototype of this was prepared by John Glasspoole, for conditions in the British Isles.* Figure 7 gives a comparison of rainfall at West Hartford during specially noteworthy years. Beside the months, the "Rafter" divisions also are given. The practically uniform average line shows that for the period (68 years), the average monthly rainfall is nearly uniform, showing slight acceleration for July and August due to thunderstorms. Attention is called to the large gain during the last of July and first of September both in the minimum year record of 1935. As these abnormalities were due to local thunderstorms, it is provocative of some thought as to what might have been the record had the unstable conditions which produced them been absent. It is a fact, however, that these sudden and more or less local storms are a feature of rainfall in this locality of consider-

TABLE 4.—MINIMUM RUN-OFF FOR CONSECUTIVE MONTHS AND CALENDAR YEARS, NEPAUG RIVER WATERSHED.

Period Years of Minimum Run-off.

No. of Consecutive Months.	Date of Period.	Average 12 Months Run-off, Inches.	Per cent. of Average Annual Run-off.
12	Mar. 1930–Feb. 1931	9.434	41.9
24	Apr. 1930–Mar. 1932	13.000	57.7
36	Jul. 1929–Jun. 1932	13.519	60.0
48	Oct. 1928–Sep. 1932	15.418	68.5
60	Jan. 1928–Dec. 1932	18.672	82.9

Period of Calender Years of Minimum Run-off.

No. of Consecutive Years.	Date of Period.	Average Yearly Run-off, Inches.	Per cent. of Average Annual Run-off.
1	1930.....	11.45	50.8
2	1930–31.....	13.45	59.7
3	1930–32.....	15.30	68.0
4	1929–32.....	16.88	75.0
5	1928–32.....	18.64	82.7

Average Annual Run-off for Years of Record (1912–1932) = 22.53 in.

*Trans. Inst. W. Engr. (Eng.) *35*, p. 174.

TABLE 5.—MINIMUM RAINFALL FOR CONSECUTIVE MONTHS AND CORRESPONDING RUN-OFF, NEPAUG RIVER WATERSHED.

No. of Consec. Months.	Date of Period.	Run-off for Period.			Rainfall for Period.			Per cent. Period Rainfall to Rainfall Years Rec'd, (2 ÷ 3).	Per cent., Col. 1, of Average Annual Rainfall.	Per cent. Collection, (4 ÷ 1).
		Total.	Inches. Per Month.	Ave. Monthly Years of Record.	Total.	Inches. Per Month.	Ave. Monthly Years of Record.			
		1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
1	Mar. 1915.....	0.13	0.13	3.47	1.28	1.28	4.19	3.6	0.3	984.6
2	Mar. –Apr. 1915....	2.18	1.09	3.60	3.02	1.51	3.99	30.3	5.0	138.5
3	Oct. –Dec. 1928....	4.44	1.48	3.63	2.06	0.69	1.54	40.8	10.2	46.4
4	Mar. –Jun. 1915....	7.29	1.82	3.69	4.91	1.23	2.96	49.3	16.7	67.4
5	Jul. –Nov. 1931....	10.56	2.11	3.83	1.22	0.24	1.03	55.1	24.2	11.6
6	Jul. –Dec. 1931....	13.43	2.24	3.73	1.82	0.30	1.14	60.1	30.8	13.6
7	Aug. 1931–Feb. 1932	17.80	2.54	3.58	4.17	0.60	1.42	70.9	40.8	23.2
8	Jul. 1931–Feb. 1932	20.43	2.65	3.59	4.63	0.58	1.34	73.8	46.9	22.7
9	Sep. 1931–May 1932	23.88	2.65	3.55	10.28	1.14	2.18	74.6	54.8	43.0
10	Aug. 1931–May 1932	27.36	2.74	3.63	10.55	1.06	2.27	75.5	62.9	38.6
11	Jul. 1931–May 1932	29.99	2.73	3.63	11.01	1.00	1.93	75.2	68.9	36.7
12	Jul. 1931–Jun. 1932	33.55	2.80	3.63	11.72	0.98	1.88	77.1	77.1	34.9
15	Sep. 1930–Nov. 1931	44.19	2.95	3.65	16.33	1.09	1.74	80.8	101.5	37.0
18	Jul. 1930–Dec. 1931	53.82	2.99	3.66	17.41	0.97	1.63	81.7	123.7	32.3
24	Jan. 1930–Dec. 1931	71.72	2.99	3.63	26.91	1.12	1.88	82.4	164.8	37.5
30	Jun. 1929–Nov. 1931	89.94	3.00	3.66	30.81	1.03	1.72	87.0	206.7	34.3
36	Jun. 1929–May 1932	109.37	3.04	3.63	25.14	0.70	1.88	83.7	251.3	23.0
48	Oct. 1928–Sep. 1932	154.24	3.21	3.63	61.56	1.28	1.88	88.6	354.4	39.9
60	Aug. 1921–Jul. 1936	205.35	3.42	3.63	102.55	1.71	1.88	94.3	471.7	49.9

Average Monthly Rainfall for Years of Record (1912–1932) = 43.52 in.
Average Monthly Run-off for Years of Record (1912–1932) = 22.53 in.

TABLE 6.—MINIMUM RAINFALL FOR CONSECUTIVE MONTHS AND CALENDAR YEARS, NEPAUG RIVER WATERSHED.

Period Years of Minimum Rainfall.

No. of Consecutive Months.	Date of Period.	Average 12 Months Rainfall, Inches.	Per cent. of Average Annual Rainfall.
12	Jul. 1931–Jun. 1932	33.5	77.1
24	Jan. 1930–Dec. 1931	35.86	82.4
36	Jun. 1929–May 1932	36.46	83.7
48	Oct. 1928–Sep. 1932	38.56	88.6
60	Aug. 1921–Jul. 1926	41.07	92.3

Period Calendar Years of Minimum Rainfall.

No. of Consecutive Years.	Years.	Average Yearly Rainfall, Inches.	Per cent. of Average Annual Rainfall.
1	1930.....	34.89	80.2
2	1930–1931.....	35.86	82.4
3	1929–1931.....	38.72	90.0
4	1928–1931.....	39.37	90.5
5	1928–1932.....	41.46	95.3

Average Annual Rainfall (1912–1932) = 43.52.

able importance, as shown by the slight upward trend of the average line at this point.

The yield (run-off) from watersheds, however, is usually of much more interest to the waterworks than rainfall, and the periods of low yield do not always coincide with those of low rainfall.

For a conception of this statement, attention is invited to Tables 3 to 6, for relationship between minimum conditions of rainfall and run-off by consecutive monthly periods on the "Nepaug" catchment area.

The matter of yield in periods of drought is intimately affected by ground water storage. This matter was discussed somewhat by the writer in a paper before the American Geophysical Union in 1936,* and Fig. 8 is taken from that discussion, in order to indicate relationship between rainfall, run-off and ground water run-off during periods of drought. Particular points illustrated by these curves are:

The effect of the low rainfall on run-off during 1931 and 1932 seems quite clear namely:

The quick response of run-off to increased rainfall immediately following the valleys.

The lag in ground water recovery to rainfall, but its quick response the first part of 1933 when rainfall definitely took an upward path.

The most important matter of all, however, is the ability of a gathering ground to maintain yield during periods of extreme drought.

*Trans. Am. Geoph. Union, Section of Hydrology, 1936.

As has been stated, the recent drought period for Hartford conditions began about the middle of 1929. (See curve of rainfall departures in Figure 8), — and continued well into 1933. The effect of these conditions on storage in the Nepaug Reservoir of the Metropolitan District is shown by the curves in Fig. 9. The Nepaug Reservoir is located about 14 miles northwest of Hartford and has a drainage area of 32 sq. mi. and a capacity of 9.54 billon gal. The full line in Fig. 9 shows the effect of draft at a rate of 25 m.g.d., while the dotted line shows the conditions of water surface if drawn on at a rate of 23 m.g.d.

Nepaug Reservoir was formed by damming two nearly parallel streams and raising the water some 30 ft. over the divide. The effluent gate house for this reservoir is located at Phelps Brook Dam and there are about 2 billion gal. of storage unavailable at the present time. It is to be noted that draft at a rate of 25 m.g.d. would have exhausted the available supply, present demand (1935) 18.24 m.g.d., and additional supply works are now under construction.

For the four calendar years, 1928–1932 inclusive, the minimum annual rainfall at Nepaug Dam was 34.89 in. and the average 38.12 in. It has been shown elsewhere that the minimum period of rainfall for this locality centered around 1881; therefore, it seems probable that lower conditions of yield for this catchment area may be expected.

Computations of probable rainfall at Nepaug, based on relationship to the long-term West Hartford record, gives for the period 1880 to 1883 inclusive, the lowest annual rainfall of 34.73 in. and a four-year average of 37.29 in. While the computed difference in the four year averages is only 0.83 in., as the earlier period was immediately preceded by a run of comparatively low rainfall years and the later period included several years of abundant rainfall, it appears reasonable to conclude that minimum yield of Nepaug water is less than 23.5 m.g.d.; for safety it has been tentatively set at 21 m.g.d.

Floods.

The second part of this discussion "Floods" usually focuses attention on a more dramatic spectacle than that of drought, and, as the poet said:* "Men may come and men may go, but I go on forever," so floods will come and floods will go and the mightiest river is but a combination of brooks. It is well therefore, to look ahead somewhat and consider what has happened in the past.

This portion of the paper will be confined principally to local matters affecting the southern part of New England first, the general matter of excess rainfall and run-off; second, the effect of flood flows on the design of the additional water supply system for the Hartford Metropolitan District, and third, the "great flood" experienced by Hartford in March 1936, both on the Farmington and Connecticut Rivers.

*"The Brook" Alfred Tennyson.

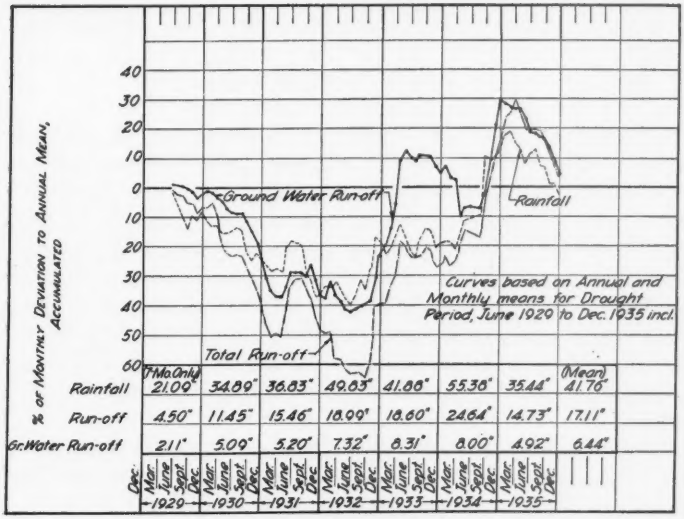

FIG. 8.— RAINFALL, RUN-OFF AND GROUND WATER RUN-OFF, NEPAUG RIVER WATERSHED.

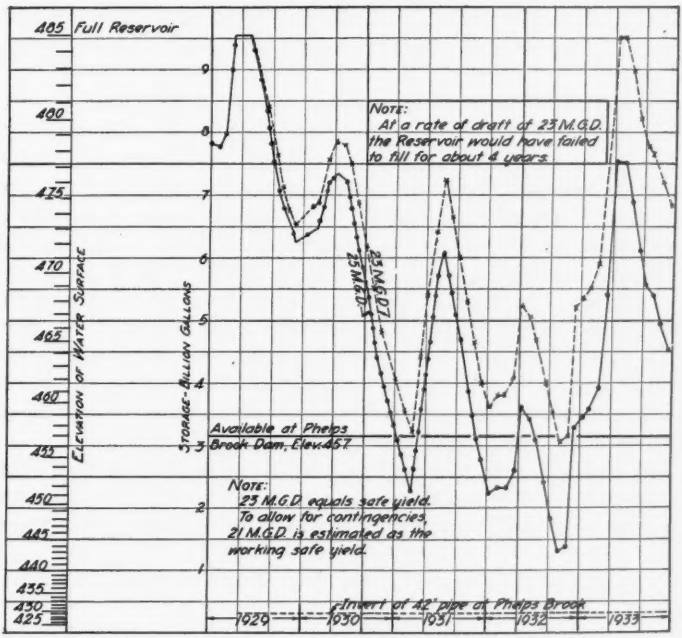

FIG. 9.— ESTIMATED DEPLETION OF NEPAUG RESERVOIR UNDER ASSUMED DRAFTS OF 23 AND 25 MILLION GALLONS PER DAY.

As with drought, floods in this section are usually brought about by vagaries of depressions that have left their usual path and in combination with other meteorological and geographical elements produce the excess rainfall that is reflected in the run-off in the streams.

It seems needless to state that much of the damage caused by floods is the result of man's encroachment on natural waterways and their bottom land which nature has provided in connection with her needs. If man could only remember this and work with, instead of against, natural conditions, he would be saved much distress to both body and property. All stream valleys show the marks of former floods, and even within the memory of living persons there have been floods as great or perhaps greater than those recently experienced under similar conditions. The fact that these earlier floods have done less damage is due probably to less confinement of the stream by encroachments and to more storage available in low lands bordering the river.

While man may not be able to foretell the combination of weather that will produce high water in the streams, it is of a surety that what has happened once will happen again and not unlikely in greater amount.

With drought, the limit is no rain, and that is final; with rain producing floods, there may be no upper limit to the amount of rain that may fall, although there are certain conditions on which it seems reasonable to predicate an assumption that would preclude such a flood and of such duration in this part of the world as brought Noah's adventures to universal attention.

Yarnell* has given many rainfall intensity frequency curves for various parts of the country, and in "Storm Rainfall of the Eastern United States†," may be found an exhaustive discussion of excessive rainfall in this part of the country. In two recent publications of the U. S. Geological Survey‡ will be found valuable and voluminous data on rainfall and flood relationships, their frequency and magnitude, together with a discussion and application of the latest methods applicable to these subjects.

As stated, New England may be considered out of the usual pathway of the "great storms," although many have come this way. Much of its area is on the leeward side of storm exposure, so that visitations of this kind are more or less of "freak" character. Nevertheless, as they have come in the past, so they will return again and again in the future, and it behooves us to "watch out" that we are not caught unaware.

As a general rule, the monthly fall of rain in southern New England is fairly uniform, although there are some wide deviations from the 68 years of record, which are due to storms of a more or less flood-producing character.

Conditions as observed at the West Hartford station are given in Table 7.

*Senior Drainage Engineer, U. S. Dept. Agri., Misc. Pub. No. 204, Aug. 1935.
†Tech. Rep., Pt. V, Miami Conservancy District (1917).
‡Water Supply Papers, Nos. 771 and 772.

TABLE 7.—MONTHLY EXTREMES AND MONTHLY AVERAGES, RAINFALL AT WEST HARTFORD, 1868–1935 INCLUSIVE.

Month	68-Yr. Aver. Inches.	Maximum Inches.	Maximum Year.	Minimum Inches.	Minimum Year.
Jan.	3.42	8.48	1891	1.00	1916
Feb.	3.31	8.28	1900	0.50	1901
Mar.	3.76	9.38	1877	0.17	1915
Apr.	3.40	11.17	1901	0.73	1896
May	3.59	9.10	1868	0.20	1887
Jun.	3.33	10.81	1903	0.15	1873
Jul.	4.00	15.14	1897	0.66	1924
Aug.	4.33	10.27	1885	0.25	1876
Sep.	3.64	10.88	1882	0.25	1881
Oct.	3 68	13.33	1869	0.18	1924
Nov.	3.62	8.30	1932	0.73	1933
Dec.	3.28	9.34	1901	0.67	1877
Total	43.36				

TABLE 8.—MAXIMUM PEAK AND 24-HOUR FLOOD FLOWS FOR EACH YEAR OF RECORD (1913–1936, INCLUSIVE), EAST BRANCH. Arranged in Order of Magnitude of Peak Flow.

No.	Year.	Date of Duration.	Flood Flow, c.f.s. per sq. mi. Peak.	Flood Flow, c.f.s. per sq. mi. 24-Hr.	Rainfall, Inches.
1	1927	Nov. 2–4	141.8	95.50	6.52
2	1936	Mar. 11–20	135.8	98.0	6.83
3	1924	Apr. 6–7	73.01	53.30	4.10
4	1928	Aug. 26	64.25	39.77	4.05
5	1932	Nov. 10–11	59.60	35.98	2.74
6	1915	Feb. 25	52.16	36.84	2.27
7	1917	Mar. 27–28	42.38	30.48	1.27
8	1913	Oct. 26–27	38.79	29.34	5.33
9	1919	May 22	37.49	29.42	3.56
10	1933	Apr. 2–3	37.34	30.89	0.93
11	1922	Mar. 7, 8	37.14	23.42	1.67
12	1920	Sept. 30–Oct. 1	35.98	29.54	4.26
13	1925	Feb. 12	34.80	31.86	2.12
14	1935	Jan. 10	34.07	22.57	2.06
15	1929	Apr. 21	32.49	24.16	1.57
16	1923	Apr. 4–5	28.48	26.90	1.27
17	1931	Jun. 10	26.90	24.16	3.40
18	1921	Mar. 9, 10	25.32	17.30	1.40
19	1916	Feb. 26	25.10	19.89	2.42
20	1914	Mar. 2	21.84	18.09	2.88
21	1930	Apr. 7	21.40	16.30	1.68
22	1934	Mar. 5, 6	17.51	15.29	1.30
23	1926	Apr. 25	16.04	12.34	1.31
24	1918	Mar. 22, 23	11.58	8.30	Melting Snow
	Total	.....	1,050.47	.....	.....
	Average	.....	43.77	.....	.....

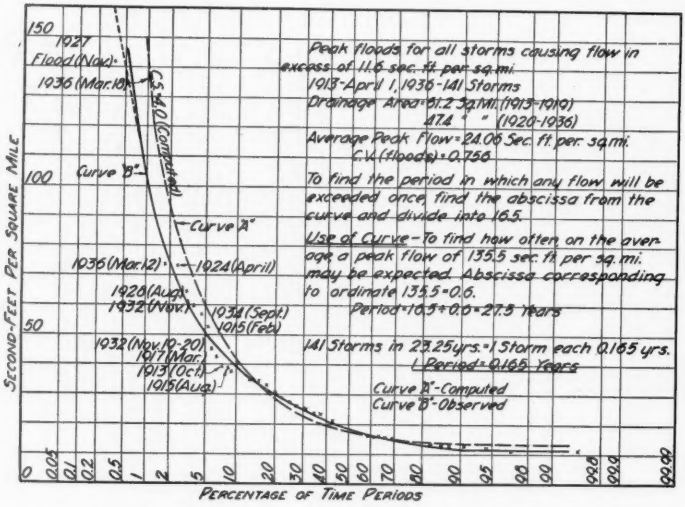
Peak floods for all storms causing flow in excess of 11.6 sec. ft. per sq. mi.
1913-April 1, 1936-141 Storms
Drainage Area-91.2 Sq. Mi. (1913-1919)
47.4 " " (1920-1936)
Average Peak Flow-24.06 Sec. ft. per sq. mi.
C.V. (Floods)-0.756
To find the period in which any flow will be exceeded once, find the abscissa from the curve and divide into 16.5.
Use of Curve-To find how often on the average a peak flow of 135.5 sec. ft. per sq. mi. may be expected. Abscissa corresponding to ordinate 135.5-0.6.
Period-16.5÷0.6-27.5 Years
141 Storms in 22.25 yrs.-1 Storm each 0.165 yrs.
Period-0.166 Years
Curve A"-Computed
Curve B"-Observed
1927 Flood (Nov.)
1936 (Mar. 18)
Curve B"
Curve A"
1936 (Mar. 12)
1924 (April)
1928 (Aug.)
1932 (Nov.)
1934 (Sept.)
1915 (Feb.)
1932 (Nov. 10-20)
1917 (Mar.)
1913 (Oct.)
1915 (Aug.)
SECOND-FEET PER SQUARE MILE
PERCENTAGE OF TIME PERIODS

FIG. 10.— DURATION CURVE OF EAST BRANCH FLOOD PEAK FLOWS.

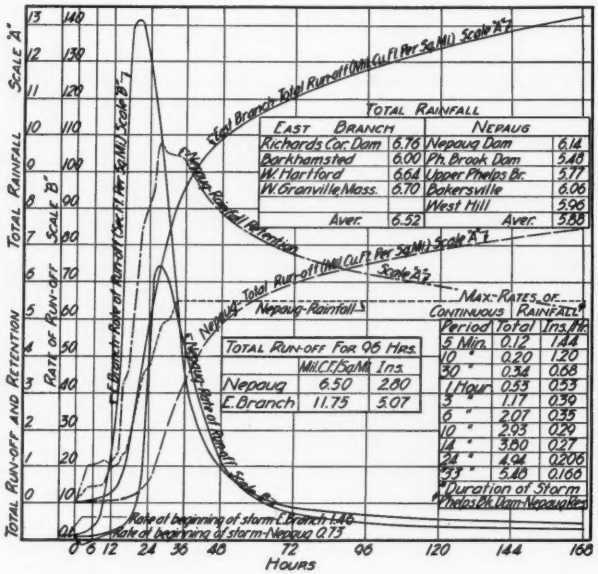

FIG. 11.— NEPAUG AND EAST BRANCH STORM CHARACTERISTICS, NOVEMBER, 1927.

In connection with the development of the additional water supply for Hartford, careful records have been kept of rainfall and run-off on two different water supply areas: On the Nepaug watershed (23.9 sq. mi.) there are 9 raingages of U. S. standard, with 1 recording gage; on the East Branch watershed (47.4 sq. mi.) there are 8 standard raingages and 2

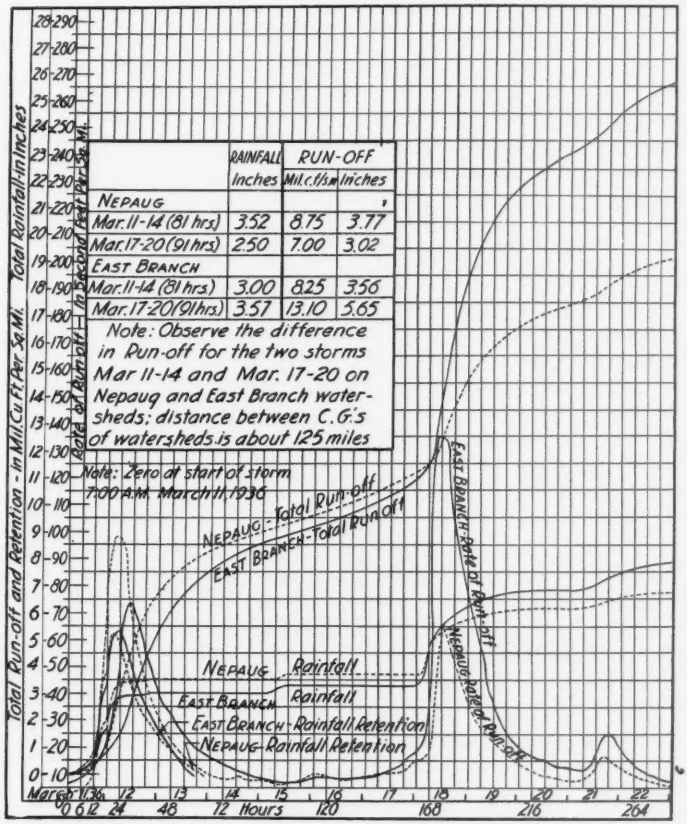
Total Run-off and Retention - in Mil. Cu. Ft. Per Sq. Mi.
Total Rainfall-in Inches
Rate of Run-off in Second Feet Per Sq. Mi.
RAINFALL Inches
RUN-OFF Mil. C.f./s.m.
RUN-OFF Inches
NEPAUG
Mar. 11-14 (81 hrs.) 3.52 8.75 3.77
Mar. 17-20 (91 hrs.) 2.50 7.00 3.02
EAST BRANCH
Mar. 11-14 (81 hrs.) 3.00 8.25 3.56
Mar. 17-20 (91 hrs.) 3.57 13.10 5.65
Note: Observe the difference in Run-off for the two storms Mar 11-14 and Mar. 17-20 on Nepaug and East Branch watersheds; distance between C.G.'s of watersheds is about 125 miles
Note: Zero at start of storm 7:00 A.M. March 11, 1936
NEPAUG - Total Run-off
EAST BRANCH - Total Run-off
EAST BRANCH - Rate of Run-off
NEPAUG - Rate of Run-off
NEPAUG Rainfall
EAST BRANCH Rainfall
EAST BRANCH - Rainfall Retention
NEPAUG - Rainfall Retention
March 11 36 12 13 14 15 16 17 18 19 20 21 22
0 6 12 24 48 72 Hours 120 168 216 264

FIG. 12.— NEPAUG AND EAST BRANCH STORM CHARACTERISTICS, MARCH, 1936.

recording gages. Nearly all of the principal streams have standard weirs with automatic recording devices.

The record of flood frequency, 1912 to 1936, on the East Branch of the Farmington River and the magnitude of these floods appear in Table 8, listed in order of magnitude for the years of record. Figure 10 has been prepared from these data, using statistical methods suggested by the late

Allen Hazen.* In the use of his method of analysis, the statement of Hazen is most pertinent, namely, that the results are better named "chance" than "probability"; that is, that instead of stating that a peak flow may be expected once in 27.5 years, it is better to state that the chance of a run-off of that magnitude is about 1 to 27.5.

Attention is called to the time of the year of the two greatest floods experienced in the Hartford water supply catchments in the years of record, 1912 to 1936. The characteristics of the storms producing the floods November, 1927, and March, 1936, appear in Figures 11 and 12. The flood of November, 1927, came in the fall of the year and was due solely to the rainfall at that time; that of March, 1936, was in the spring of the year and beside its producing rainfall, the run-off was increased by the winter's accumulation of snow and ice on the watershed.

The conditions of the March, 1936, storm are estimated to be: —

East Branch Watershed (62 sq. mi.)
Rainfall (March 11 to noon March 26) 8.14 in.
Run-off (March 11 to noon March 26) 27,255,000 cu. ft. per sq. mi., or 11.7 in.
Peak flow 135 c.f.s. per sq. mi.
Nepaug Watershed (32 sq. mi.)
Rainfall (March 11 to noon March 26) 6.95 in.
Run-off (March 11 to noon March 26) 20,442,000 cu. ft. per sq. mi., or 8.8 in.
Peak flow 98 c.f.s. per sq. mi.

The areas subjected to flood-producing rainfall in several of the "great" storms that have plagued northeastern U. S. within the past 75 years are given in Table 9.†

TABLE 9 — SUMMARY OF RAINFALL-AREA RELATIONS FOR NEW ENGLAND.
Area (square miles) covered by rainfall in inches.

DATE OF STORM.	RAINFALL (Inches).								
	4	5	6	7	8	9	10	11	12
Oct. 3– 4, 1869*	.....	.....	13,570	7,150	2,260	902	370	255	40
Feb. 10–14, 1886*	.....	4,850	3,000	1,500	750	.....	.....	.....	.....
Oct. 12–14, 1895*	.....	6,340	4,070	2,040	590	.....	.....	.....	.....
Nov. 3– 4. 1927*	.....	22,030	8,470	4,040	1,795	497	.....	.....	.....
Mar. 9–21, 1936†	30,721	22,422	14,639	7,783	4,510	2,087	773	129	.....

*From "Report of Committee on Floods," p. 342. *Jour'l of Boston S. C. E.*, Vol. XVII, No. 7.
†U. S. D. A., Weather Bureau, New England Section, March, 1936 and Acc. Ge 2079.

The relationship of three of these great storms to the Hartford district is shown in Figure 13. Suffice it to state that the storm of October 3–4, 1869,‡ which centered with its 12.5 in. of downpour within the area at present under development for water supply and a similar storm with 12.13

*Flood Flows, John Wiley & Sons.

†Storm March 1936 from U. S. Dept. Agr. Wea. Bu., "Climatological Data, N. E. Section. March 1936 believed to be tentative only and subject to revision and extension. The other data from this JOURNAL 44, 1930, p. 119 et seq.

‡See this JOURNAL, March 1925, map opposite 0p. 18.

in. had its focus* near Westerly, Rhode Island, September 16, 1932, about 80 miles east of Barkhamsted.

The Hartford Metropolitan District now has under construction a reservoir on the East Branch of the Farmington River at Barkhamsted, which when completed will have a capacity of 30 billion gal. or 92,000 acre-feet, a water surface of 3.6 sq. mi. and a drainage area of 52.7 sq. mi. above the dam.

The dam is of earth with masonry core wall 136 ft. high above original river bed. Below this reservoir is another of 3-billion gal. capacity and below that several populous manufacturing communities of high property value.

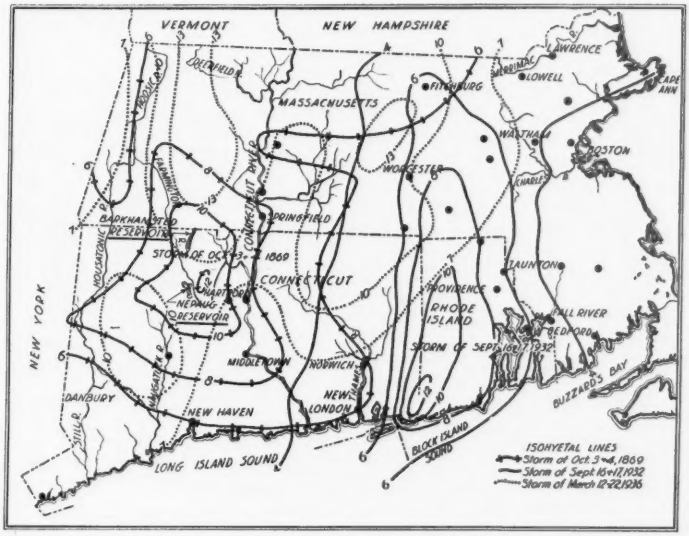

FIG. 13.— ISOHYETAL LINES OF THREE GREAT STORMS.

One of the most important requirements connected with the design of this reservoir was to give absolute protection to these places, and most careful study was given to protection against disastrous floods. The computated discharges at the spillway were checked by means of model study at the Worcester Polytechnic Institute, under direction of Prof. C. M. Allen. Many studies were made in order to determine what might be the reasonable discharge from this area with focus similar to those of 1868 (Canton) and 1932 (Westerly, R. I.,) shown in Fig. 13, and effects of the later storms of November, 1927, and March, 1936, were checked against the results of these studies. For this purpose the unit graph method of L. K. Sherman† was used to a considerable extent, and the finally adopted results were based to a large degree on his methods.

*Journ. Am. Meteor. Soc. Jan. 1933, *14*, p. 25.
†*Eng. News-Record, 108* (1932), p. 501, also *U. S. Geol. Surv. Water Supply Paper*, No. 772, p. 123.

The final hydrograph developed and used in the design is shown in Fig. 14, in which also have been plotted, for purposes of comparison, hydrographs from the highest flood peaks observed on this stream during the period of record (1912 to 1936). Figures 11 and 12 show the rainfall and run-off characteristics for the storms of 1927 and 1936. In passing, it seems pertinent to remark that these two storms seem characteristic of

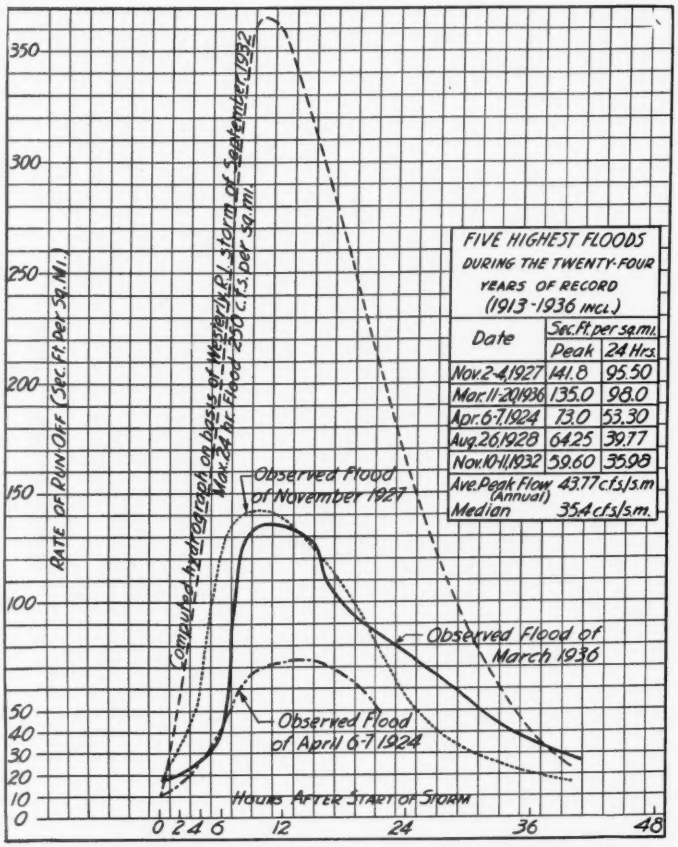

FIG. 14.—FLOOD HYDROGRAPHS OF EAST BRANCH OF FARMINGTON RIVER.

the time of the year in which they occurred. The first (1927) fall storm, had a single peak; that is, the storm came and went by in about 48 hours, and its result was a single item in the river history. The results of the second stormy period produced two peaks in the river record, the second one of much greater magnitude than the first, due to the first storm having loosened up the ice and snow on the watershed and leaving it in good con-

dition for run-off when the second storm occurred. In the fall of the year a rainfall like to that of March 1936 probably would have produced a run-off similar to that of 1927; that is, it would have been over within 48 hours or so. The second storm of the 1936 period, an inch less in amount, except for the ice and snow accompaniment, probably would have been remarked only as exceedingly high water.

The flood peak eventually provided for in the new work was 500 c.f.s. per sq. mi., equivalent to a run-off on the watershed of 26 500 c.f.s., of which 15 000 c.f.s. would pass over the spillway under a head of 7½ ft. and the remainder temporarily stored in the surcharge capacity of the reservoir, Fig. 14, providing for a chance expectancy of about 1 to 1 000. The check of this figure against several well established formulas is given in Table 10.

TABLE 10.—SUMMARY OF PEAK FLOOD FLOWS.
Watershed area, 53 sq. mi. Spillway Elevation, 530.0 ft. Freeboard, 15 ft.

Formula.	Peak Flows, c.f.s. per sq. mi.	Reservoir Level	Spillway Discharge c.f.s. per sq. mi
Boston Society of Civil Engineers			
1927 flood basis			
6-in. run-off	194	533.6	90
8-in. run-off	258	534.5	130
1936 flood basis			
6-in. run-off	173	533.3	80
8-in. run-off	231	534.1	110
Kuichling			
Frequent floods	217	533.9	105
Rare floods	422	536.5	235
Pettis			
Probable 100-yr. flood	330	535.4	175
Probable 1 000-yr. flood	396	536.2	215
Probable 10 000-yr. flood	429	536.6	240
Probable 20 000-yr. flood	445	536.8	250
Metcalf and Eddy			
By formula	151	532.9	65
By curve of observed storms (88 in number)	680	541.8	350
Boston Society of Civil Engineers			
Westerly, R. I., flood 1932, 10.19 in. run-off (flood)	365	535.8	195
Fuller			
Probable 100-yr. flood	120	532.5	52
Probable 500-yr. flood	147	532.9	64
Probable 1 000-yr. flood	158	533.1	70
Probable 1 500-yr. flood	165	533.2	74
Probable 2 500-yr. flood	171	533.2	75

From this it appears that even with the very rare storm, mentioned by Metcalf and Eddy*, with the reservoir full, the addition due to the stated flood would bring high water to a point about 3 ft. below the top of the embankment, while the Westerly, R. I. storm of September, 1932, (12.13

*Am. Sewerage Practice, Vol. 1, 2nd Ed. 1928 (p. 316).

in.), the chance run-off of which is estimated to be about as 1 to 300, would produce a head of about 6 ft. on the spillway with 9-ft. freeboard, and a discharge at a rate of 200 c.f.s. per sq. mi. against an estimated spillway-channel capacity of 340 c.f.s. per sq. mi. It so happened that topographical conditions lent themselves very readily to this large spillway capacity without undue cost.

It is a coincidence only that the run-off provided for on the new water supply work happens to have a computed frequency of occurrence similar to that of the run-off in the Connecticut River for March 1936. There was no connection, and the computations were made and model work done several years previous to 1936.

The Great Flood of 1936.

Coming now to the flood of March 1936, it is proposed to consider its characteristics and effects, first on the Farmington River and next on the Connecticut River, particularly as relates to Hartford.

Farmington River. The two branch streams that form the Farmington River, have their rise in western Massachusetts and join at New Hartford about 15 miles south of the Massachusetts-Connecticut line, about 2 miles below Greenwoods Dam on the West Branch. The flow of the stream is southerly to Farmington, where it receives the tributary Pequabuck from the west (58 sq. mi.), a distance of about 15 miles below New Hartford; thence northeasterly for about 14 miles to Tariffville where the Salmon Brook tributary joins from the north (area 75 sq. mi.); thence southeasterly into the Connecticut River at Windsor, about 6 miles north of Hartford. It has a total drainage area at its junction with Connecticut River of about 600 sq. mi.; about 150 sq. mi. of this area are in Massachusetts and the remainder in Connecticut. The elevation of the sources of the West and East Branches is about 1 400 ft. above mean sea level in Otis Reservoir and Noyes Pond respectively. Following down the West Branch which is the principal stream, the elevation at Massachusetts-Connecticut State line is 650; at the junction of the two branches 345; at Farmington, El. 160; at Tarriffville, El. 130; and at the junction with the Connecticut River, about El. 20. At Collinsville, about 9 miles north of Farmington (tributary area 360 sq. mi.), it receives the discharge of the Nepaug watershed from the west (32 sq. mi.), and 5 miles farther north the discharge of the East Branch watershed from the east (62 sq. miles).

The storm rainfall began on March 11, 1936, and the daily amounts are tabulated in Fig. 12. The flood hydrograph for New Boston,* Riverton† (located on West Branch) and Tariffville‡ show the progress of the flood downstream (Fig. 15).

*U. S. Geol. Surv., No. Atlantic Sect., H. B. Kennison, Dist. Engr.
†These figures from 10th U. S. Census (1880), Reports on Water Power, Part 1, p. 249.
‡U. S. Geol. Surv., No. Atlantic Sect., B. L. Bigwood, Dist. Engr.

At New Hartford was the Greenwoods industrial development of Landers, Frary & Clark, with an earth dam about 35 ft. high and about 400 ft. long, forming a pond of about 125 acres, and with a capacity below spillway of about 550 acre-feet (180 m.g.). The spillway of the dam is said to have been from 210 to 230 ft. in length and 25 feet high, a rockfilled timber structure built in 1848 and referred to as "having been well designed and constructed."*

The most spectacular single event of the "great flood" (March, 1936), in the Farmington River was the washout of the spillway portion of the

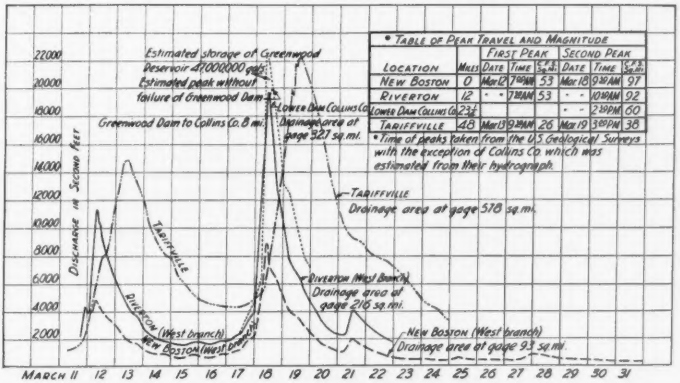
* Table of Peak Travel and Magnitude
Location | Miles | First Peak Date | Time | C.F.S. per Sq. mi. | Second Peak Date | Time | C.F.S. per Sq. mi.
New Boston | 0 | Mar 17 | 7:00 AM | 53 | Mar 18 | 9:30 AM | 97
Riverton | 12 | " " | 7:30 AM | 53 | " " | 10:00 AM | 92
Lower Dam Collins Co. | 23½ | | | | " " | 2:30 PM | 60
Tariffville | 48 | Mar 19 | 9:30 AM | 26 | Mar 19 | 3:00 PM | 38
* Time of peaks taken from the U.S. Geological Surveys with the exception of Collins Co. which was estimated from their hydrograph.
Estimated storage of Greenwood Reservoir 470000000 gals.
Estimated peak without failure of Greenwood Dam
Greenwood Dam to Collins Co. 8 mi.
Lower Dam Collins Co. Drainage area at gage 327 sq. mi.
Tariffville Drainage area at gage 578 sq. mi.
Riverton (West branch) Drainage area at gage 216 sq. mi.
New Boston (West branch) Drainage area at gage 93 sq. mi.
Discharge in Second Feet
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
March 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31

FIG. 15.— PROGRESS OF MARCH, 1936, FLOOD PEAK ALONG FARMINGTON RIVER.

dam at Greenwoods (New Hartford, Conn.). Description of the breaking of this dam is given below, and the effect on the hydrograph at Collins Co. (Collinsville, Conn.) lower dam, about 8 miles below Greenwoods, appears in Fig. 15.

An eye witness, Mr. F. O. Church of New Hartford, living on the west bank of the river about 1 000 ft. below the dam, stated that about 8 A.M. of March 18, (1936), he noticed some large logs floating past his home. It was raining heavily at the time, and the river was badly swollen. At about 8:15, he noticed the east end of the spillway giving way; the river water suddenly rose and he had barely time to get his automobile from the basement and to the State Highway in front of his home, which stands 20 ft., or more, above the river bed. In a short time the whole spillway went out, and the flow of water reached his house. Mr. Church estimated that perhaps 15 minutes elapsed from the time he saw the first break until the spillway was entirely washed out, and very shortly thereafter the river reached its maximum height (8:30 to 8:45 A.M.), and remained practically at crest for perhaps an hour before beginning to recede.

*10th U. S. Census—Report on Water Power in New England.

It is estimated that there were about 470 m.g. (or 1 500 acre-feet) in storage, including surcharge in Greenwoods Pond. The head on the spillway at the time of break was about 7.0 ft. Interesting accompaniments of this flood were the effects at the new reservoir now under construction at Barkhamsted (Bills Brook Dam) for the Metropolitan District, on the East Branch about 3 miles above the junction of the East and West Branches. Here the embankment was completed entirely across the valley, and the stream flowing freely (automatic discharge) through the dam in a concrete conduit of horse-shoe shape, 21 ft. horizontal and 17 ft.

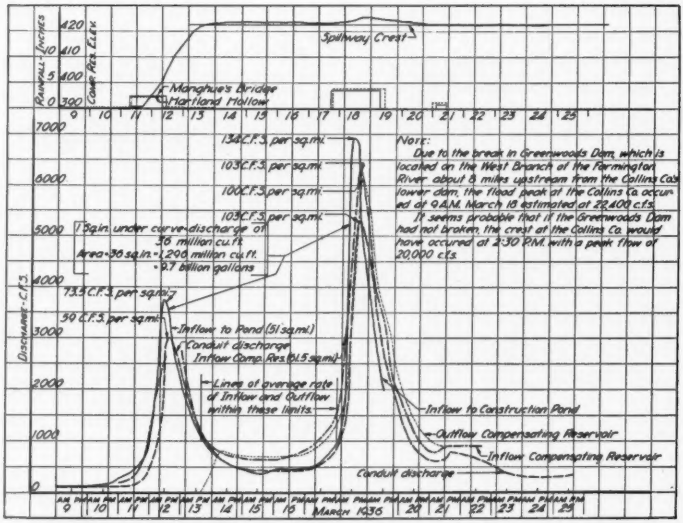
Rainfall - Inches
Comp. Res. Elev.
Spillway Crest
Maughie's Bridge
Hartland Hollow
Discharge - C.F.S.
134 C.F.S. per sq.mi.
103 C.F.S. per sq.mi.
100 C.F.S. per sq.mi.
103 C.F.S. per sq.mi.
Note:
Due to the break in Greenwoods Dam, which is located on the West Branch of the Farmington River about 6 miles upstream from the Collins Co. lower dam, the flood peak at the Collins Co. occurred at 9 A.M. March 19 estimated at 22,400 c.f.s.
It seems probable that if the Greenwoods Dam had not broken, the crest at the Collins Co. would have occurred at 2:30 P.M. with a peak flow of 20,000 c.f.s.
Area = 30 sq.in. = 1,206 million cu.ft.
= 9.7 billion gallons
73.5 C.F.S. per sq.mi.
30 C.F.S. per sq.mi.
Inflow to Pond (51 sq.mi.)
Conduit discharge
Inflow Comp. Res. (61.5 sq.mi.)
Lines of average rate of Inflow and Outflow within these limits
Inflow to Construction Pond
Outflow Compensating Reservoir
Inflow Compensating Reservoir
Conduit discharge
March 1936

FIG. 16.— EAST BRANCH HYDROGRAPH FOR FLOOD OF MARCH, 1936.

vertical, with a bell mouth entrance. This conduit discharged into a lower reservoir (compensating), where at the time there was available below the spillway storage of 7 900 acre feet (total capacity 9 000 acre feet). The peak outfall over this spillway reached a maximum of about 3 ft. The storage here and that in the pond above the new dam somewhat delayed the second flood peak immediately below Richards Corner Dam, but it appears to have been of slight effect, if any, on the lower part of the river due to valley storage.

The hydrograph from the East Branch, Fig. 16, shows two peaks due to rainfall conditions, one on March 12, the other on March 18. On account of the available storage in the Compensating Reservoir, the discharge over the East Branch spillway due to the first flood flow did not begin until about 4 P.M. March 13, although there was some discharge previously through the waste gates. When the second peak came, however,

the Compensating Reservoir was full and all the space available for the flood was in the surcharge, that is, above spillway level.

The second, or principal, peak passed Richards Corner Dam (Compensating Reservoir) about 7 P.M., March 18, Fig. 16. The peak in the main river at Collinsville Dam occurred at about 9 A.M., on the same day, due to the break in the dam at Greenwoods. Eliminating this happening under ordinary flood conditions, it is estimated that the peak in the main river probably would have reached Collinsville at about 2:30 P.M., March 18. Without storage on the East Branch or the conditions due to the break at Greenwoods, it seems probable that the combined flood peak of both branches would have reached Collinsville about 12:30 P.M., March 18.

These flood peaks, however, were not synchronous at Collinsville, because of the storage in the East Branch, but about two hours apart. Due to channel storage below the confluence of the two branches and particularly that above Satan's Kingdom gorge, it is questionable, however, if the storage on the East Branch would be of much effort at Collinsville. Although the storage on the East Branch is considerable compared with its area, the East Branch area itself is only about one-fifth of the total flood-producing area at Collinsville. In this connection, it is noteworthy to consider the effect of valley storage and tributary area in plotting flood peaks, as well as the uncertain value of reservoir storage on water level in streams.

During the flood of November, 1927, with only a little storage on the East Branch and flood flows of that stream not much different from those of March, 1936, the peak of the 1936 flood was only about 0.3 ft. lower than that of 1927 at Collinsville. The condition of flood heights at Collinsville was confirmed by observations at the Farmington River Power Company's plant 29 miles down the river, where it is said that the flow during the 1927 flood was 23 800 c.f.s., and that for the 1936 flood a little less. While peak discharge at Collinsville in the flood of 1936 was about 20 000 c.f.s. from 327 sq. mi. that at Tariffville, 25 miles below, was at a rate of only 22 000 c.f.s. from a tributary area of 578 sq. mi.; the 24-hr. maximum flow at Tariffville was 20 500 c.f.s. compared with 16 140 c.f.s. at Collinsville.

These conditions seem in accord with the statements of Arthur E. Morgan,* former Chief Engineer of the Miami Conservancy District (Dayton Flood Protection), and those of Col. Kelley,† U. S. Engineer Corps, namely, that reservoirs for efficient flood protection should be located immediately above the area to be protected, Mr. Morgan stating "The writer fully agrees with Col. Kelley that no reservoirs on the distant head waters of tributaries to the Mississippi can have any appreciable effect on Mississippi floods."

*Transactions, Am. Soc. C. E., *93*, (1929), p. 752.
†Trans. Am. Soc. C. E., *93* (1929), pp. 724–754.

From these statements and local observation, it appears that local protection probably is more effective than valley storage on tributary streams a hundred miles or so north of the threatened area, as well as avoiding interstate complications if a general flood protection scheme is proposed. The progress of the flood peak in the main Farmington River is shown in Fig. 15. The time of passage of water in the Farmington River is as follows:—

	Distance, Miles.	Ordinary Conditions, Mi. per Hr.	Flood Conditions, Mi. Per Hr.
New Boston to Riverton* (West Branch)	12	..	12
Greenwoods Pond (West Branch) to Collinsville.	8	2	8
Compensating Reservoir (East Branch) to Collinsville	7	2	7

Table 11 gives the annual maximum flood flows on the East Branch arranged in order of 24-hr. magnitude for the 24 years of record.

TABLE 11.—MAXIMUM 24-HOUR FLOOD FLOWS FOR EACH YEAR OF RECORD (1913—THROUGH MARCH 1936), EAST BRANCH.
Arranged in order of Magnitude of 24-Hour Floods.

Order of Magn.	C.f.s. per Sq. Mi.	Prob. of Occ. (24 hr.) Once In—	Corr'pdg Peak Flow, c.f.s. per Sq. Mi.	Prob. of Occ. (Peak) Once In—	Ratio (4 ÷ 2).	Inches of Rainfall.	Durat'n of Rainfall in Hrs.	Average Rate Rainfall (7 ÷ 8) per Hr.	Max. Rate Rainfall per Hr. at Any Hr.	Date.
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*
Greatest	98.0	35.7	135.5	32.1	138.3	6.83	53	.129	.42	Mar. 18, 1936
2	95.5	33.3	141.8	32.5	148.5	6.52	33	.198	.59	Nov. 2–4, 1937
3	53.3	28.2	73.0	24.4	137.0	4.10	15	.274	.52	Apr. 6–7, 1924
4	39.77	25.3	64.2	23.3	161.4	4.05	26	.156	.66	Aug. 26, 1928
5	36.84	24.4	52.2	20.9	141.7	2.27	18	.126	..	Feb. 25, 1915
6	35.98	24.1	59.6	22.3	165.6	2.74	17	.161	.37	Nov. 10–11, 1932
7	31.86	23.5	34.8	18.0	109.2	2.12	40	.053	.29	Feb. 12, 1925
8	30.48	23.3	42.4	19.6	139.1	1.27	..	...	..	Mar. 27–28, 1917
9	29.54	23.0	36.0	18.2	121.9	4.26	29	.147	.65	Sep. 30–Oct. 1, 1920
10	29.42	22.9	37.5	18.5	127.4	3.56	24	.148	.53	May 22, 1919
11	29.34	22.8	38.8	18.9	132.2	5.33	50	.106	..	Oct. 26, 27, 1913
12	28.60	22.2	37.3	18.2	130.4	0.66	12	.055	..	Mar. 5, 1934
13	26.90	22.1	28.5	16.7	105.9	1.27	..	...	..	Apr. 4, 5, 1923
14	24.16	21.3	32.5	17.5	134.5	1.57	48	0.33	.20	Apr. 21, 1929
15	24.16	21.3	26.9	16.3	111.3	3.40	48	.071	.27	Jun. 10, 1931
16	23.42	21.1	37.1	18.5	158.6	1.67	7	.239	..	Mar. 7, 8, 1922
17	22.60	20.9	34.1	17.9	150.8	2.52	11	.229	..	Jan. 10, 1935
18	19.89	20.2	25.1	15.8	126.2	2.42	..	...	..	Feb. 26, 1916
19	18.10	19.6	25.9	16.0	143.1	1.47	12	.123	.22	Sep. 16, 1933
20	18.09	19.6	21.84	14.9	120.7	2.88	..	...	..	Mar. 2, 1914
21	17.30	19.3	25.32	15.8	146.4	1.40	12	.117	..	Mar. 9, 10, 1921
22	16.30	18.9	21.40	14.7	131.3	1.68	12	.140	.40	Apr. 7, 1930
23	12.34	17.3	16.04	13.2	130.0	1.31	4	.327	.66	Apr. 25, 1926
Least	8.30	15.7	11.58	11.8	139.5	Melting Snow	..	...	..	Mar. 22, 23, 1918

*U. S. Geol. Survey Hydrograph.

For purposes of comparison of conditions on different rivers, attention is called to the characteristic "K" described by Gourley.* The characteristic K for the East Branch, as determined at the Bills Brook Dam is 0.418 based on four year averages (1922–1935).

$$\left(\frac{\text{Average daily flow below average}}{\text{Average daily flow}}\right)$$

The following tabulation shows the relationship of area to peak discharge during the flood period on the Farmington River:

River	Station	Drainage Area, Sq. Mi.	Maximum Flows—c.f.s. 24 Hour Total.	24 Hour Per Sq. Mi.	Peak. Total	Peak. Per Sq. Mi.
East Branch	Manghue's Bridge.....	47.4	4,650	98	6,400	135
West Branch	New Boston.....	93	6,350	68	9,000	97
West Branch	Riverton.....	216	13,980	65	19,800	92
Farmington	Collins Co. (lower dam)	327	16,140	49	20,000	60
Farmington	Tariffville.....	578	20,550	36	22,200	38

The Connecticut River Flood of Hartford, Conn. Hartford, Conn. has now passed through a flood period during which the Connecticut River reached its highest discharge during nearly 100 years of record (1843–1936) and received therefrom damage estimated at the present time to amount to about seven million dollars.

For the meteorological details of the storm producing this flood, it is probable that the U. S. Weather Bureau will issue some reliable statement of facts which will appear in some of their publications when the data have been collected and analyzed.

Floods due to rainfall only, and fall floods particularly, are usually of short duration with high peak flows lasting only a few hours. On small watersheds, great floods are due to what usually are designated "cloud bursts" — on the larger watersheds, these floods are usually caused by long-continued rains of less intensity, and the cumulative effects of tributary discharge. Small floods occur annually, moderately high floods at intervals of several years, and great floods are of rare occurrence.

Considering the Hartford record of high water in the Connecticut River in the 93 years of record (1843–1936 inclusive), the median height is 20.7; *i.e.*, that height above and below which there are the same number of flood peaks. This may perhaps be taken as the limit of *small* floods in the total number of annual floods. In the period of record there have been peaks between gage heights of 22 and 27.5 seventy-seven times, and in view of the character of the damage ensuing, it would seem reasonable to say that peaks above gage height 20.7 and not exceeding 27.5 might be classed as *moderate*, 32 of these peaks reached gage height 22; 29 reached gage height 22.5; 14 gage height 25 and 4 up to gage height 27.5. Above gage height 27.5 to gage height 37.5 reached by the flood of March 1936,

*This JOURNAL, *60* (1936).

TABLE 12.—ANNUAL FLOODS, CONNECTICUT RIVER—GAGE HEIGHTS—IN ORDER OF MAGNITUDE (1843*–1936). Drainage area at Hartford—10 560 sq. mi.

No.	Date.	Gage Height, Ft.	Flood Flow.	
			Total, c.f.s.	C.f.s. per Sq. Mi.
1	Mar. 1936	37.6	300 000	28.4
2	1854	29.8	205 000	19.5
3	Nov. 1927	29.0	196 000	18.6
4	1862	28.7	192 000	18.2
5	1843	27.2	175 000	16.6
6	1896	26.5	170 000	16.1
7	1869	26.5	167 000	15.8
8	Apr. 1901	26.4	169 000	16.0
9	1859	26.4	166 000	15.7
10	Mar. 1913	26.3	165 000	15.6
11	Apr. 1933	26.0	161 000	15.3
12	1895	25.7	161 000	15.3
13	Mar. 1902	25.5	159 000	15.1
14	1870	25.3	154 000	14.6
15	Apr. 1909	24.7	150 000	14.2
16	1865	24.7	147 000	13.9
17	Apr. 1922	24.5	145 000	13.7
18	1893	24.0	143 000	13.6
19	Apr. 1905	24.0	143 000	13.6
20	1878	23.9	142 000	13.5
21	1874	23.8	137 000	13.0
22	Feb. 1900	23.4	136 000	12.9
23	Mar. 1903	23.3	135 000	12.8
24	1856	23.3	132 000	12.5
25	1852	23.1	130 000	12.3
26	Apr. 1934	23.1	130 000	12.3
27	1877	22.8	130 000	12.3
28	1887	22.5	127 000	12.0
29	Mar. 1920	22.5	127 000	12.0
30	Apr. 1932	22.2	121 000	11.5
31	1899	22.0	122 000	11.6
32	Apr. 1923	22.0	119 000	11.3
33	1876	21.9	121 000	11.5
34	1884	21.9	121 000	11.5
35	Apr. 1914	21.9	121 000	11.5
36	1886	21.7	119 000	11.3
37	1861	21.5	115 000	10.9
38	1868	21.5	115 000	10.9
39	1879	21.4	116 000	11.0
40	1898	21.2	114 000	10.8
41	Apr. 1912	21.2	114 000	10.8
42	1847	21.0	110 000	10.4
43	1873	20.9	109 000	10.3
44	1897	20.8	111 000	10.5
45	Apr. 1916	20.8	111 000	10.5
46	Apr. 1926	20.8	111 000	10.5
47	Nov. 1907	20.7	110 000	10.4

*No record for 1892. Note: Zero of gage 1.57 ft. above Hartford City Base.

TABLE 12.—ANNUAL FLOODS, CONNECTICUT RIVER, GAGE HEIGHTS—IN ORDER OF MAGNITUDE (1843*–1936).—Continued.
Drainage area at Hartford, 10 560 sq. mi.

No.	Date.	Gage Height, Ft.	Flood Flow. Total, c.f.s.	C.f.s. per Sq. Mi.
48	Apr. 1924	20.7	110 000	10.4
49	1850	20.7	107 000	10.2
50	Feb. 1915	20.6	106 000	10.0
51	1883	20.5	108 000	10.3
52	Mar. 1925	20.5	108 000	10.3
53	Jan. 1935	20.5	108 000	10.3
54	1853	20.5	106 000	10.1
55	1866	20.5	106 000	10.0
56	1867	20.0	101 000	9.6
57	Jan. 1910	20.0	101 000	9.6
58	Mar. 1921	19.9	100 000	9.5
59	1891	19.8	102 000	9.8
60	May 1906	19.8	102 000	9.8
61	Mar. 1919	19.8	100 000	9.5
62	1872	19.7	99 000	9.4
63	1844	19.5	97 000	9.2
64	1857	19.5	97 000	9.2
65	1888	19.4	98 000	9.3
66	Apr. 1904	19.2	97 000	9.2
67	1845	19.0	93 000	8.8
68	Mar. 1929	18.9	92 000	8.7
69	Apr. 1918	18.8	91 000	8.6
70	1846	18.7	90 000	8.5
71	Apr. 1928	18.7	90 000	8.5
72	1871	18.5	89 000	8.4
73	1875	18.4	90 000	8.6
74	Mar. 1917	18.3	87 000	8.2
75	1885	18.1	88 000	8.4
76	Mar. 1908	18.1	88 000	8.3
77	Apr. 1931	18.0	85 000	8.1
78	1849	17.5	81 000	7.7
79	1864	17.2	79 000	7.5
80	1881	16.4	75 000	7.1
81	1890	16.0	73 000	6.9
82	1860	16.0	71 000	6.7
83	1889	15.6	70 000	6.6
84	1848	15.5	68 000	6.5
85	Apr. 1911	15.5	68 000	6.5
86	1880	15.0	66 000	6.3
87	1855	15.0	64 000	6.1
88	1863	15.0	64 000	6.1
89	1882	14.7	64 000	6.1
90	1851	14.5	61 000	5.8
91	Apr. 1930	14.2	60 000	5.7
92	1894	13.8	59 000	5.6
93	1858	12.2	48 000	4.6

*No record for 1892. Note: Zero of gage 1.57 ft. above Hartford City Base.

there have been 4 recorded floods. Floods therefore above gage height 27.5 may be classed as *great* floods.

From statistical analysis, it would appear that these limit floods include magnitudes that might be expected over a very long period, the lower limit with an average frequency of perhaps once in about 20 years, a chance of the November, 1927, flood height of about once in 25 years, and of the flood of March, 1936, of about once in 300 years.

In Table 12, are given the recorded flood heights in the Connecticut River in order of magnitude from the greatest to the least, and in Figure 17, is shown the result of a statistical analysis of these data.

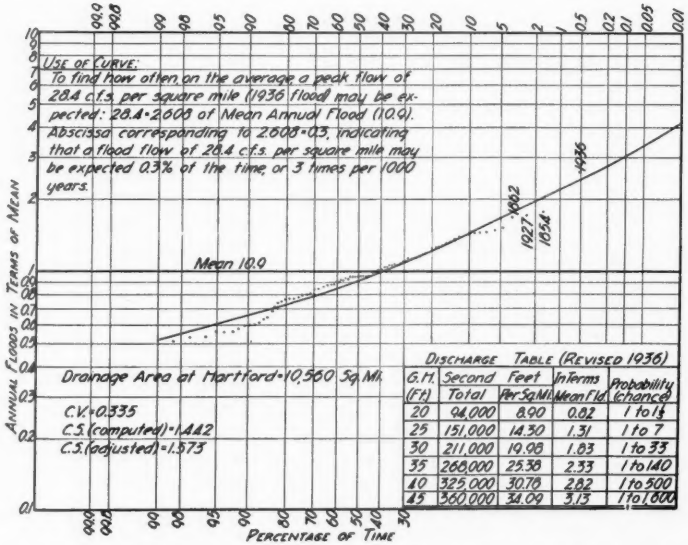
Use of Curve:
To find how often on the average a peak flow of 28.4 c.f.s. per square mile (1936 flood) may be expected: 28.4÷2.603 of Mean Annual Flood (10.9).
Abscissa corresponding to 2.603=.03, indicating that a flood flow of 28.4 c.f.s. per square mile may be expected 0.3% of the time, or 3 times per 1000 years.
Mean 10.9
1936
1927
1954
1902
Drainage Area at Hartford=10,560 Sq. Mi.
C.V.=0.335
C.S.(computed)=1.442
C.S.(adjusted)=1.573
Discharge Table (Revised 1936)
G.H. (Ft.) | Second Feet Total | Second Feet Per Sq. Mi. | In Terms Mean Fld | Probability (chance)
20 | 94,000 | 8.90 | 0.82 | 1 to 1¼
25 | 151,000 | 14.30 | 1.31 | 1 to 7
30 | 211,000 | 19.98 | 1.83 | 1 to 33
35 | 268,000 | 25.38 | 2.33 | 1 to 140
40 | 325,000 | 30.78 | 2.82 | 1 to 500
45 | 380,000 | 34.09 | 3.13 | 1 to 1,600
Annual Floods in Terms of Mean
Percentage of Time

FIG. 17.—PROBABILITY CURVE OF ANNUAL CONNECTICUT RIVER FLOODS AT HARTFORD.

It must be realized, however, that the occurrences which produced the flood peak of March 1936 are not susceptible of rigid statistical analysis on account of the character of the data. Here were two storms happening to follow each other in quick succession at a time when there were large quantities of snow and ice on the watershed, a combination of circumstances that the record itself shows to be extremely rare.

On the question of protection from floods of this character, that is a matter for local decision as to method proposed, and for careful consideration aided by competent engineering advice as to plans and estimates and the amount of money involved. The expediency of the expenditure, however, of the investment balanced against the chance of a peak flood of unusual height is a business problem. The matter is one of insurance with

the cost of maintenace and annual fixed payment on 40-year serial bonds equivalent to the premium, balanced against value destroyed by floods from Connecticut, combined with back water from Park River.

The total damage to Hartford, including land, buildings, merchandise, loss of business and cost of cleaning up, has been estimated at $7 000 000.*

It is presumed that this figure does not recognize the improved condition of many buildings in the flooded area due to cleaning and replacements, much of which had been sorely needed for many years previously. While statistical analysis indicates a chance of one in about 300 years, the combination of several of the meteorological phenomena that happened to synchronize at the time indicates probability of even a much greater chance. As Hartford is an insurance center, there are here many

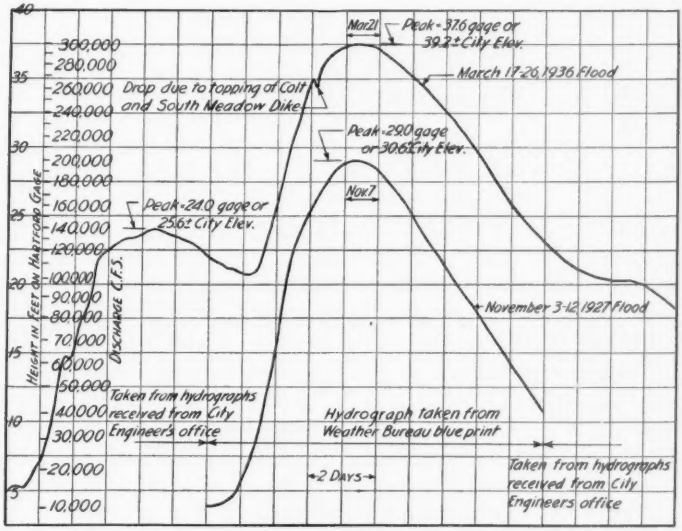

FIG. 18.— HYDROGRAPH OF CONNECTICUT RIVER AT HARTFORD FOR NOVEMBER, 1927, AND MARCH, 1936, FLOODS.

able actuaries who can give the best answer to this matter from a statistical viewpoint, and the City Board of Finance or that of the Metropolitan District can decide as to the expediency of the investment from point of view of protection only.

For purposes of record there are given the hydrographs of the periods of November 1927 and March 1936 in Fig. 18,† a list of annual flood peaks

*Since this paper was prepared, it is stated in the public press (*Hartford Courant*, September 2,1936) that it has been decided to accept U. S. Government relief funds to the amount of $250,000 and raise Clark Dike. This will protect the Hartford airport (Brainard Field) in which the city has a considerable investment at no cost (?), but leave the problem of protection to the business part of Hartford still open to solution.

†Replotted from diagram supplied by Hartford Office U. S. Weather Bureau.

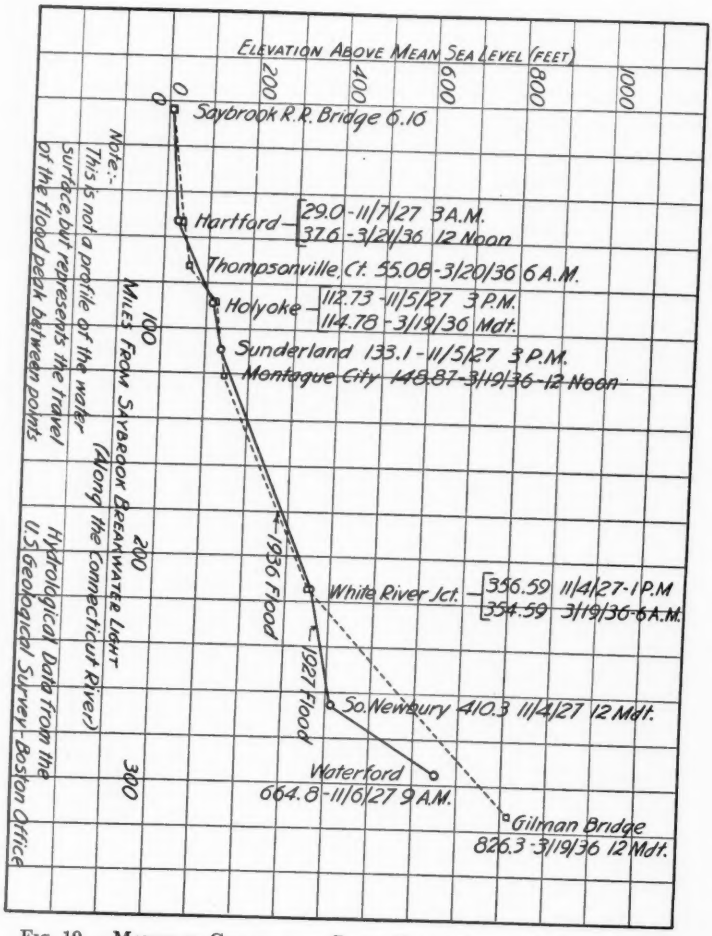
ELEVATION ABOVE MEAN SEA LEVEL (FEET)
0
200
400
600
800
1000
Saybrook R.R. Bridge 6.16
Hartford — 29.0 - 11/7/27 3 A.M.
37.6 - 3/21/36 12 Noon
Thompsonville, Ct. 55.08 - 3/20/36 6 A.M.
Holyoke — 112.73 - 11/5/27 3 P.M.
114.78 - 3/19/36 Mdt.
Sunderland 133.1 - 11/5/27 3 P.M.
Montague City 148.87 - 3/19/36 - 12 Noon
1936 Flood
White River Jct. — 356.59 11/4/27 - 1 P.M
354.59 3/19/36 - 6 A.M.
1927 Flood
So. Newbury 410.3 11/4/27 12 Mdt.
Waterford
664.8 - 11/6/27 9 A.M.
Gilman Bridge
826.3 - 3/19/36 12 Mdt.
100
200
300
MILES FROM SAYBROOK BREAKWATER LIGHT
(Along the Connecticut River)
Note:- This is not a profile of the water surface, but represents the travel of the flood peak between points
Hydrological Data from the U.S. Geological Survey - Boston Office

FIG. 19.— MARCH OF CONNECTICUT RIVER FLOOD PEAKS, NOVEMBER, 1927, AND MARCH, 1936.

in order of magnitude in Table 12, and the march of the flood peaks for 1927 and 1936, plotted from present information, in Fig. 19.*

It may be remarked that the flood of March 1936 had two peaks, the first of which was due to the storm of March 11 and 12 inclusive, and the second peak due to a second storm of March 17 to 19 inclusive.

A glance at the hydrograph will show that if the second storm had not followed so closely, say it had been delayed 3 or 4 days, it is probable that neither storm would have produced a flood of sufficient magnitude to have caused damage. It must also be considered, however, that if the second storm had followed the first more closely than it did; that is, if the barometric depression which produced this storm had arrived somewhat earlier, even the flood peak of March 21 might have been exceeded and a very much greater part of the center of Hartford might have been under water.

Thus the matter of chance of occurrence comes in, and no human skill has yet been able to foretell such happenings. The late Allen Hazen once observed "that we have come to believe that there is no such thing as a maximum flood." It may be pertinent to quote also H. M. Chittenden,† "that man is to a large extent the author of his own misfortunes in the matter of flood destructiveness, by his encroachments on the natural areas which nature has provided for her flood use."

What is stated regarding encroachments on the flood channels of rivers and thereby inviting damage to life and property by attempting to thwart Nature's courses may be applied with equal force to excessive drought damage experiences in some other parts of the country. Due to governmental enthusiasm for making "the desert blossom as the rose" certain parts of this country have been opened up to occupancy that apparently Nature in this period of world history had not intended for that purpose, and has lured thereto, settlers, who, if willing to work, could have maintained self respect and a comfortable income by struggling elsewhere with conditions similar to those of New England. Again the pertinent question: Is it profitable "to kick against the pricks" of natural conditions, instead of working in harmony?

In connection with the plan for flood control by reservoirs on the upper reaches of tributary streams, the following statement is of interest, taken from the "Report of the New Hampshire Water Resources Board on the March 1936 Flood"; namely, that, "although the peak discharges on the smaller streams were greater during the 1927 flood than in 1936, the larger rivers showed substantially higher peak flows in 1936." This bears out the statement in the Transactions of the American Society of Civil Engineers‡ that the flood control works of the Dayton (Ohio) Conservancy District not only might not be of much value in reducing flood

*Compiled from data furnished by district office of the U. S. Geol. Surv. and Engineers Corps, U. S. A.
†Corps of Engr. U. S. A.
‡*93*, (1929).

flow in the Mississippi at Cairo, but might under certain conditions actually be of disadvantage.

For many years the low parts of Hartford, lying along the west bank of the Connecticut River, have been protected by the Colt Dike, so-called, from Col. Colt of "firearms fame," and since 1932 by the construction of the Clark Dike for the protection of low lands south of the Colt area. As, however, such a flood height as recently experienced has not been approached within 8 ft. during the 93 years of record, it would appear that the height of Colt Dike was a reasonably safe "bet," and that of Clark Dike 2 ft. higher gave a good "factor of safety." The top elevation of Colt Dike was at El. 31.5 above mean sea level and of Clark Dike at El. 33.5 — the former was overtopped but stood firm, — Clark Dike was overtopped and breached, but the principal damage to the city occurred with the overtopping of the Colt Dike, which was 2 ft. lower than the Clark Dike. The Clark Dike happening is indicated by the notch "A" in the hydrograph, Fig. 18. The extreme height shown on the hydrograph was at El. 37.6, or about 4 ft. above that of Clark Dike. For purpose of comparison, the hydrograph of the November 3, 1927, is plotted on the same diagram with its peak corresponding in location. It is estimated that 2 900 acres were flooded in Hartford with about 46 000 acre-feet (15 000 m.g.) of water. (Fig. 20.)

As a matter for speculation only, the question is of interest as to what might have been the character of the second flood peak of March 1936 had the two storms producing this flood occurred somewhat further apart in time. No guess is hazarded as to peak flow at Hartford had these storms happened with a shorter interval. However, it appears probable that without the culmination of the periods of occurrence of the storm of March 8–14 and that of March 17–23 (see Fig. 1), there would probably have been no damaging flood, and river conditions of March 1936 would have gone down in history only as another period of excessively high water.

The flood of March 1936 is estimated at its peak to have had a discharge of about 300 000 c.f.s. (28.4 c.f.s. per sq. mi.), that of its next competitor in the years of record was 7.8 ft. less in height with a discharge of 205 000 c.f.s. (19.5 c.f.s. per sq. mi.) followed in November, 1927, with a discharge at a rate of 196 000 c.f.s., and a gage height 8 ft. lower than the flood of 1936.

To visualize the quantity of water passing during the flood, the following relations are given:

1. During the 24 hours of maximum flow, there was passed down the river at Hartford sufficient water to have covered Hartford (17 sq. mi.) to a depth of 54 ft.; the Metropolitan District (100 sq. mi.) to a depth of 9 ft.; and the State of Connecticut (5 000 sq. mi.) to a depth of 2 in. The entire flood flow would have covered the State of Connecticut to a depth of 2 ft.

2. By another analogy it is estimated that sufficient water passed to have filled the Barkhamsted Reservoir, now under construction, 65 times and to have supplied Hartford Metropolitan District requirements for 300 years at present rate of demand.

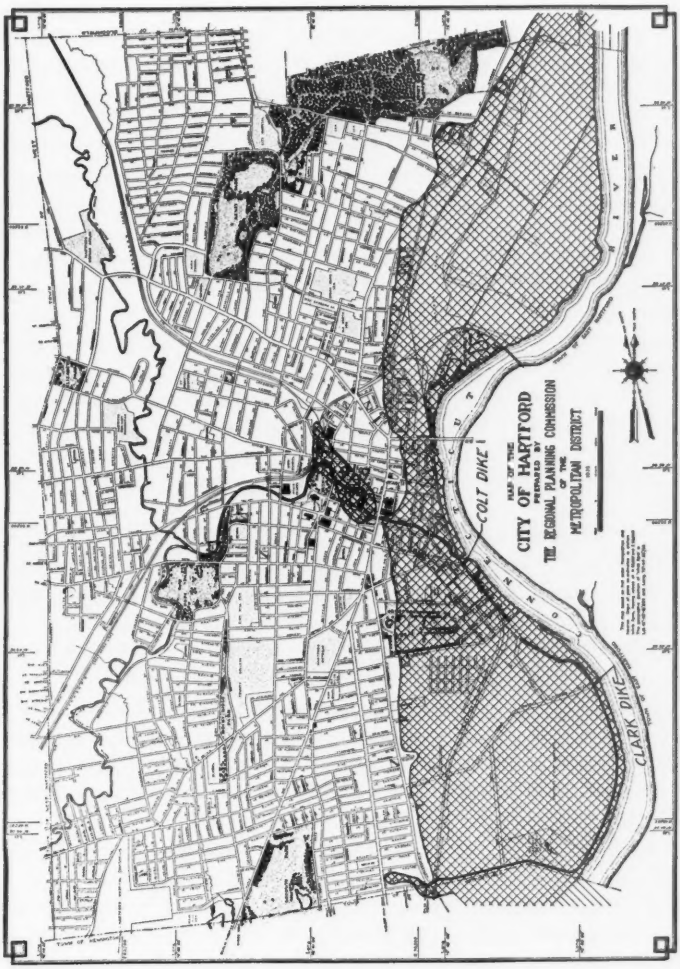

FIG. 20.— PORTION OF HARTFORD, CONN., INUNDATED IN MARCH, 1936, FLOOD.

The total capacity of the ten flood control reservoirs, proposed for immediate construction on the upper tributaries of the Connecticut River, is 352 500 acre-feet*. Data for this report were assembled and the report made up on the basis of the November 1927 flood, and submitted prior to the flood of March 1936. It is estimated from the Connecticut River hydrograph that 587 000 acre-feet, or 56 acre-feet per square mile, passed Hartford during the maximum 24-hour period, and a total of 6 million acre-feet during the flood period from midnight March 11 to noon April 2. The total combined capacities of the proposed flood reservoirs therefore are equal to about $\frac{3}{5}$ of the amount of water which passed Hartford during the maximum 24 hours of the March flood and only about 1/16 of the total water passing during the entire flood period.

It is estimated from the hydrograph of the Farmington River at Tariffville that 40 800 acre-feet were discharged from that river during a similar maximum 24-hour period, which is equivalent to 72 acre-feet per square mile, and a total of 265 000 acre-feet during the flood period from noon March 11 to noon March 25. The Barkhamsted, Nepaug and Compensating reservoirs all together hold about one-half this amount.

It is worth noting that, although the flood of 1936 was the greatest that Hartford had of record, that the amount contributed from the White River and above probably was less than from the same locality in the flood of 1927.

The drainage area of the Connecticut River at Hartford is estimated at 10 560 sq. mi. From the Metcalf and Eddy report,† it appears that in the 30 years, 1896 to 1927, there were only 5 days with gage height in excess of 27.6; *i.e.*, 10 ft. below the 1936 crest.

In the 1936 flood the lowest business part of Hartford was covered to a maximum depth of some 21 ft. with water, the waterworks shops and other buildings in this locality, near Charter Oak Avenue, had water to the middle of their second story windows, and the Southern New England Telephone Exchange, within a 1 000 ft. of the congested business center, had a depth of 7 ft. of water on its first floor. It is impossible to describe the desolate condition of this flooded area after being cleared of water; only a slight idea of it can be had from photographs taken at the time. The damage, however, was done and Hartford in its lower parts has emerged with a cleaner and more substantial section. While the present condition is a great improvement over that before the flood in many respects, the remedy is hardly to be recommended for general use. There should be other ways, less drastic, for accomplishing the purpose, but it is doubtful if it is possible to apply them under American conditions, except by such a ruthless master as this flood.

Due to the flood, there was a loss from broken pipes and fixtures of about 11.5 m.g.d. within the flooded area and continuing over about a

*House Document No. 412 of the 74th Congressional Session.
†Disposal of the Sewage from Hartford and Vicinity, 1930.

week. All services broken were sterilized with chlorine before being re-connected. Several main pipes were broken and about 3 000 ft. of 8-in. pipe were laid over ground for temporary service.

A comparison of the night water consumption from the Venturi meter charts for February 29, 1936, a day of minimum draft, March 22, a day of peak flow due to leakage on account of flood conditions, and July 8, a day of maximum summer draft, shows values of 8 m.g.d., 20 m.g.d. and 11 m.g.d. respectively.

Having cleaned up the city, the problem vividly before the citizens of Hartford was the best means of preventing a like disaster in the future. With the catastrophe underway, the wildest schemes for future safety to life and property in the city were eagerly seized upon and urged for immediate undertaking, by some even from whose position and standing in the community somewhat more of conservatism would be expected. As time passed, however, cooler judgment was exercised, and it seems probable that before funds are expended in large amounts, proper investigations will be made and conservative decision take the place of snap judgment and desire for publicity.

Three principal methods have been proposed, all of which seem to have some degree of merit, and it is expected each will be given careful thought before the final plan is adopted:

1. The method of flood control reservoirs on the upper tributaries of the river in Vermont and New Hampshire. This plan was offered by the U. S. Army Engineers after the flood of November 1927, and is said now to have their preference with modifications based on the later experience.
2. The widening of the river channel at "the narrows" near Middletown and the construction of adequate by-pass channels, preferably through ancient river cutoffs.
3. The raising of the dikes on the Hartford river front.

To all of these there are both objections and points in favor, and it may be that a combination of any or all will be the final solution of the problem. It is probable, however, that no one scheme of flood prevention or protection is a panacea for all the ills to which this locality is heir on account of floods. To any or all of them there are local conditions that complicate the matter to an extent hardly realized by the public.

Some of the objections to the reservoir plan as proposed are patent to the ordinary observer, and for all, the question of cost and its worth is one of vital importance. It may be that after careful consideration of all of the facts and proposed remedies, it will appear that, "the game is not worth the candle," so far as complete protection is concerned, but that some work may be undertaken that will afford a little more protection than now available without entering upon a project that ultimately would entail vast sums of money for construction and maintenance with no surety in sight that the remedy applied on the one hand will be even partially efficient, or on the other will be called into action at such remote periods as to make

its cost seem ridiculous. In regard to floods and their cure, it is of a surety that, "Zeus does not ratify all the designs of men."*

Note. The data received from government sources are all tentative and subject to correction by the authorities who kindly furnished them. The data have been gratefully received and are understood to be of a preliminary nature and not fully checked.

The utmost courtesy and coöperation was extended by the Corps of Engineers, U. S. A., Providence, R. I. office, Lt. Col. Mason J. Young; The U. S. Weather Bureau, both its Washington office and Hartford observer (Mr. L. F. Conover, Meteorologist and Station Director); and the U. S. Geological Survey, Water Resources, the Washington office itself and Messrs. H. B. Kinnison and B. L. Bigwood, District Engineers.

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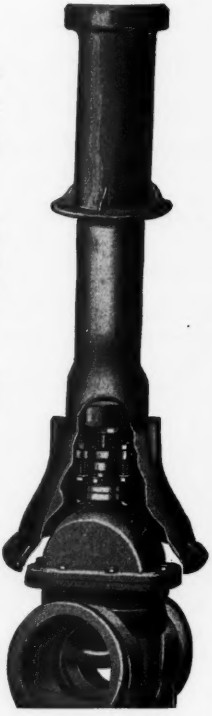

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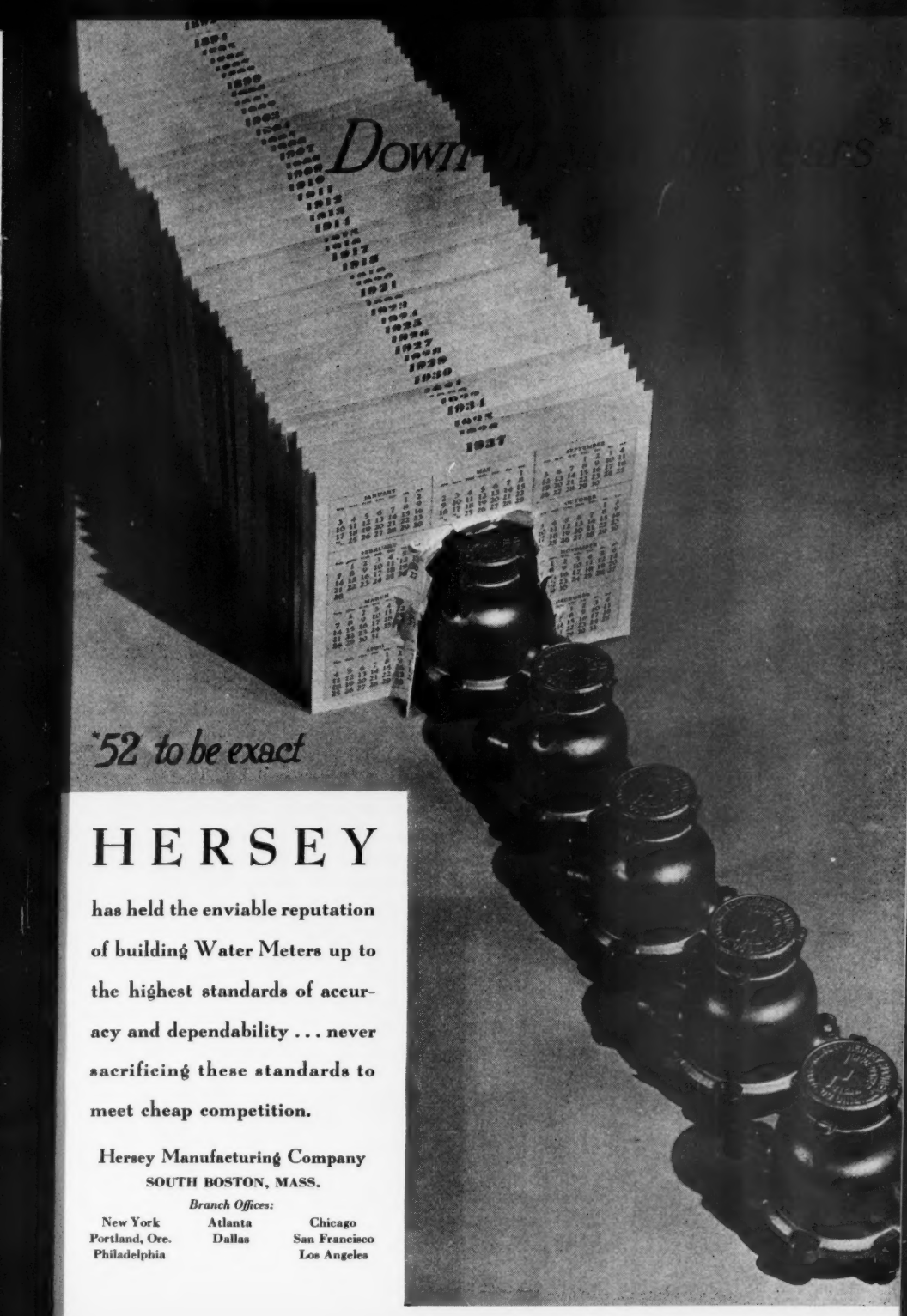

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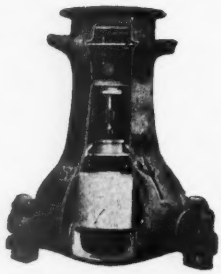

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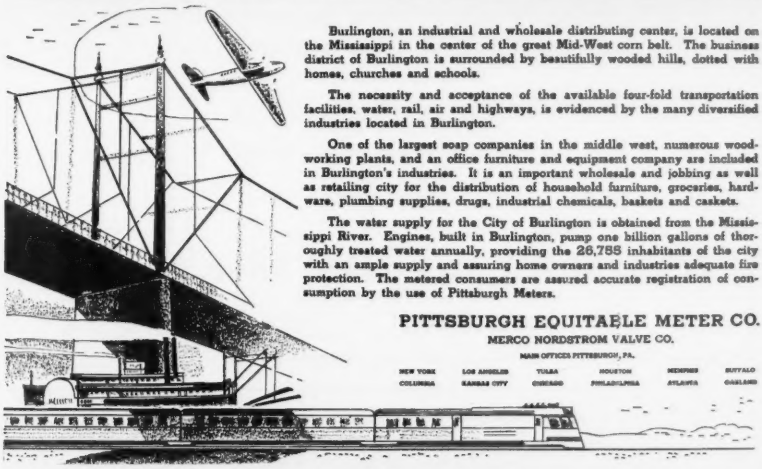

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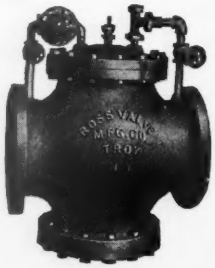

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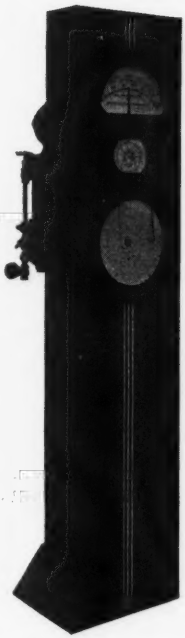

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Herschel, in the description of his epoch-making tests, remarkably visualized a broad field of usefulness for his invention. Today, however, he would express astonishment over the achievements of control, chemical proportioning, transmission of flow readings for long distances, multiple summation, and master supervision which employ the Venturi Tube as a primary element.

Builders Iron Foundry is proud of its part in this service to mankind.

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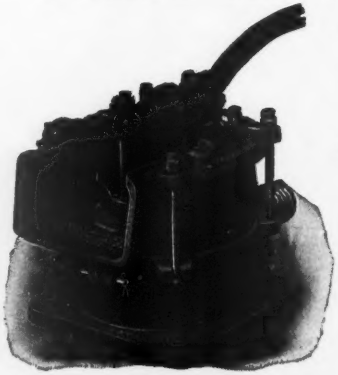
EDSON
PUMPS and ACCESSORIES



Hand Pumps – Power Pumps
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Edson Special Suction Hose
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Strainers, Adapters, etc.

Distributors for
Pollard Pipe Line Equipment
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THE EDSON CORPORATION
49 D Street, Tel. SOUth Boston 3041 South Boston
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ALSO
Warren Pipe Co. of Mass., Inc.

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CAST IRON PIPE

Flanged Pipe ***Flexible Joint Pipe***
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WORKS: PHILLIPSBURG, N. J. and EVERETT, MASS.

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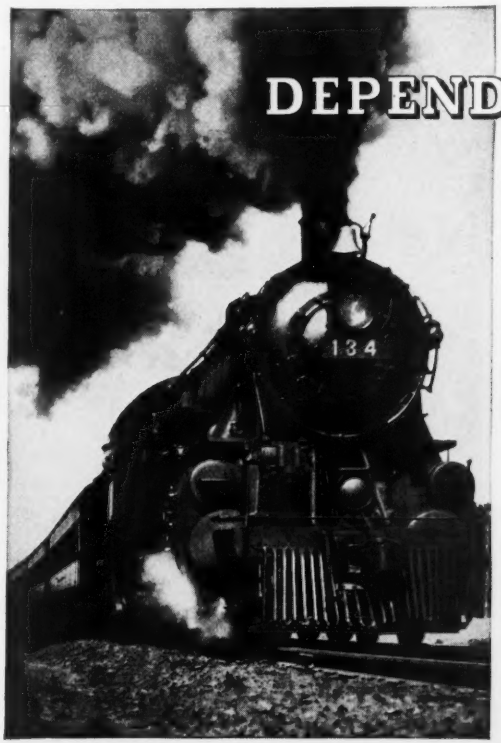


DEPENDABILITY

★

Like the American Railroad — Swift and secure — Darling Products are known for DEPENDABILITY

★

USERS of Darling Valves and Fire Hydrants have cause to be pleased with Darling dependability. Hard service through the years—without costly repairs—means reduced maintenance costs. Consistent dependability insures economy.

Quality materials, sturdy construction and skillful workmanship are the basis of ***Darling dependability.*** Further information will gladly be furnished by the nearest Darling representative. We shall be glad to consult with you on ***your particular problems.***

DARLING VALVE & MFG. CO.

Williamsport, Pa.

Representatives in

New York Philadelphia
Oklahoma City Houston

DARLING
GATE VALVES *and*
FIRE HYDRANTS

Darling Valves are the choice of those who insist on PERFORMANCE.

The height of the Darling Fire Hydrant can be adjusted to the requirements of each installation. There are many other important advantages.

"Never a Failure in Over 22 Years!"

WE like to talk about saving in material and labor—the strong, tight, flexible joints made with Hydro-Tite—the thousands of miles of pipe jointed with it.

More important, perhaps, is the fact that with hundreds of users—each laying pipe under different conditions—not a single one has reported a Hydro-Tite failure.

We make it very easy for you to change over to Hydro-Tite. Our Data Book outlines a novel proposition. Write for it.

A Symbol of Quality

A Dependable Self-Caulking Joint Compound

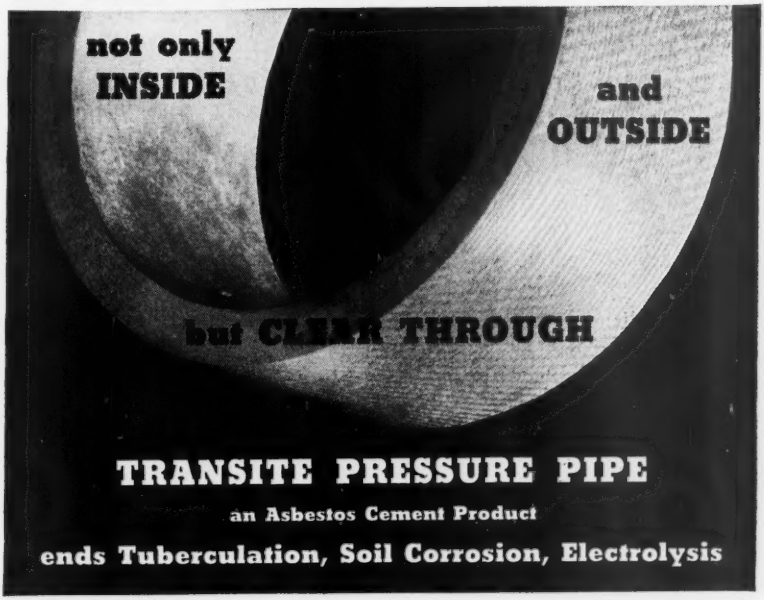
HYDRO-TITE

HYDRAULIC DEVELOPMENT CORPORATION

Main Sales Office: 50 Church Street, New York, N. Y.
General Offices and Works: West Medford Station, Boston, Mass.

DESIGNED TO END CORROSION TROUBLES

Built for Permanence



YOU dig a trench. Lay your water pipe. And hope that all will be well. But no one knows better than you that thousands of miles of water pipe have been destroyed or made ineffective by the three destructive agencies—tuberculation, soil corrosion and electrolysis.

And if you want to know the odds against *your* particular line, you have available countless engineering reports and technical papers that cover the subject thoroughly; the reduced pressures, increased pumping and maintenance costs charged against tuberculation; the enormous replacement bill charged against soil corrosion and electrolysis.

Transite Pipe, because it is made of asbestos fibre and Portland cement, is remarkably resistant to destructive agencies—and not only inside and outside but clear through.

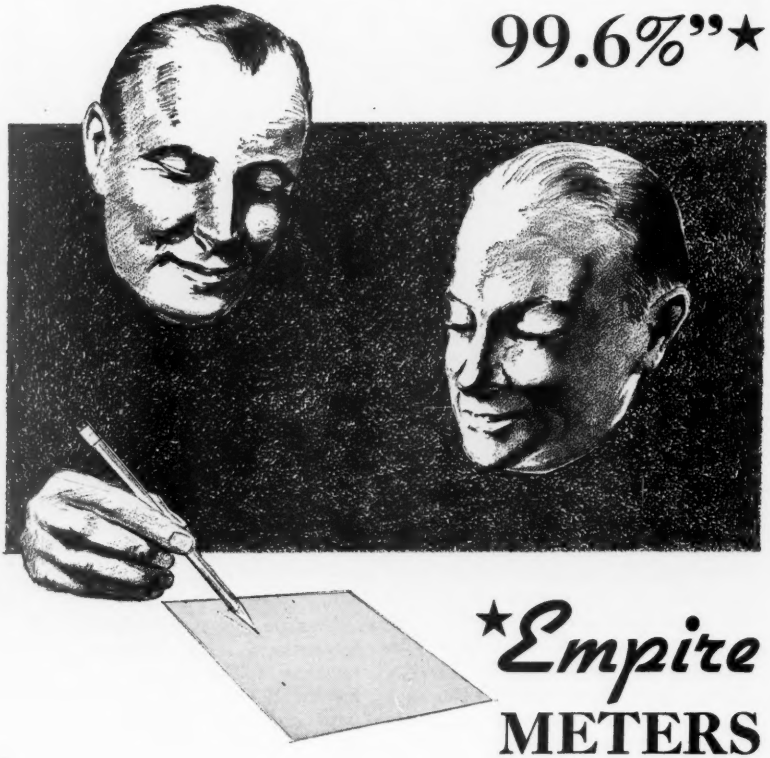
Transite Pipe is non-metallic; hence, tuberculation is a physical impossibility. Ordinary soil acids and salts cannot harm it. In all types of soil, under a wide variety of climatic conditions, it has been unaffected by soil corrosion. A non-conductor of electricity, it provides absolute protection against electrolysis.

Our new brochure gives complete engineering details. Write Johns-Manville, 22 East 40th Street, New York City.

Johns-Manville

TRANSITE PRESSURE PIPE

"Every meter tested on South Street had an average of 99.6%"★



★*Empire* METERS

NATIONAL *Meter* COMPANY

4209 FIRST AVENUE
BROOKLYN, N. Y.

BOSTON · CHICAGO · DALLAS
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Exhibiting at the New England Water Works Association Convention

SERVICE

150 Styles, Sizes and Lengths of Curb and Valve Boxes always in stock.

90% of orders are shipped the same day as received.

Extension pieces for Service, Roadway and Valve Boxes. ADAPTORS for Valve Boxes—When road surface is being raised 1 to 4 inches, don't dig—Insert Caldwell ADAPTORS.

A-34 No. 1
Stocked in 4 to 6 ft. lengths

A-38

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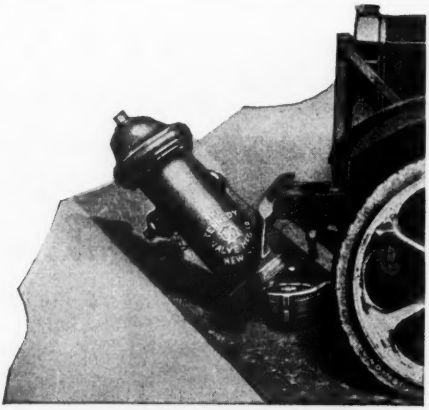
Stocked in 4 to 7 ft. lengths

SERVICE in shipments *SERVICE in merchandise*

GEORGE A. CALDWELL CO.

BOSTON MATTAPAN SQUARE MASS.

A Kennedy Safetop may be DOWN . . .



but it's never OUT of the ground

A KENNEDY SAFETOP Fire Hydrant may be broken by a colliding vehicle—but only when the impact is so severe that *no* hydrant could withstand it.

However, breakage of a Kennedy SAFETOP never involves excavation for repairs. The only parts that will be damaged are an inexpensive standpipe ring and stem coupling, *both of which are above the ground.* They cost only $6.00, and can be replaced by one man, working with a few standard tools, in less than half an hour, and without shutting off the water pressure.

Moreover, a Kennedy SAFETOP does not require excavation for adjustment of nozzle position, for change in number or size of nozzles, for lengthening to meet change in street grade, or for attention to operating parts. Once installed, a Kennedy SAFETOP stays in the ground *permanently* and never requires digging up the pavement for any purpose.

Send for Bulletins

The Kennedy Valve Mfg. Co., Elmira, N.Y.

KENNEDY

SAFETOP FIRE HYDRANT

REG. U.S. PAT. OFF.

EDDY

Valves and Hydrants

Distinctive Features of the EDDY Valves

ONLY three parts are moved by the Stem—the ball and the two gates. Each gate is hung from one of the trunnions on the ball. A convex surface at the base of each trunnion fits a concave surface on the back of the gate. This allows the gates to adjust themselves properly to their seats. . . . The gates, being free to revolve on the trunnions, do not always seat in the same position. Gates are center bearing and adjustable. They are forced to their seats with equal pressure at all points. There are two hooks on the ball on sizes 4-inch and larger, which loosely engage with the gates.

Distinctive Features of Our New Swivel Top Hydrants

THIS new hydrant has all the advantages of the popular EDDY fire hydrant, plus several additional features. . . . Nozzles are in a short flanged section of the standpipe, which may be unbolted and turned to different positions. . . . Should the standpipe be broken, only the casting below the swivel head need be replaced. . . . To raise the hydrant to conform to a new grading, simply insert a flanged extension piece below the swivelhead. No digging. To add a steamer nozzle at any time, it is only necessary to replace the swivel head with one having a steamer nozzle.

Besides valves and hydrants to meet the most exacting demands of waterworks service, the complete EDDY Line includes: valves for sewage disposal works; valves for steam, gas and oil; check valves, foot valves, plug valves, shear gates, indicator posts, etc. EDDY valves are made in three classes: Iron Body Bronze Mounted, All Bronze, and All Iron Valves.

Eddy Valve Co.

WATERFORD NEW YORK

LEAD PIPE

Used by leading WATER WORKS for over 50 years. If proper weight is used, combined with good workmanship, your FINAL COST is assured, as REPAIRS are unnecessary. LEAD PIPE is sufficiently ductile to conform to any sagging condition. There is no RUSTING of pipe or joints; no DISCOLORATION of water.

LEAD WOOL

Every Atom Pure Lead

For calking pipe joints under the most difficult conditions; for overhead joints or in wet places where the use of molten lead is not only impracticable but dangerous — LEAD WOOL may be used to advantage.

It makes an absolutely tight joint which will withstand the highest pressure, yet be sufficiently elastic to allow considerable sagging or settling of the pipe, without danger of leaks.

Compared with the poured joint, in this respect, the superiority of LEAD WOOL is apparent.

LEAD PIPE TIN-LINED LEAD PIPE
PURE BLOCK TIN PIPE SOLDER
DUTCH BOY WHITE LEAD & RED LEAD

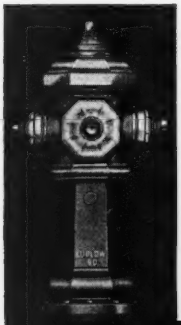
Highest Quality Soft Virgin Omaha Pig Lead

NATIONAL-BOSTON LEAD CO.

800 Albany St. Boston, Mass.

Check LUDLOW Hydrants

Check these points about the Ludlow No. 90 hydrant—



- ✓ Full force — no obstruction in waterway
- ✓ Non-freezing—drains from bottom.
- ✓ Surge and water hammer eliminated.
- ✓ Replaceable without digging.
- ✓ Parts easily removable — low maintenance.

—and finally, the most modern, graceful design obtainable. Send for free folder.

The LUDLOW VALVE MFG. CO.
TROY, NEW YORK

PUMPS { CENTRIFUGAL, STEAM, TRIPLEX, ROTARY
POWER { DIESEL, GASOLINE, STEAM, ELECTRIC
PARDEE CHLORINATORS { DIRECT, SOLUTION, VACUUM
PORTABLE { LABOUR SELF-PRIMING PUMP, AIR COMPRESSORS AND TOOLS

For Rent or Sale

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PUMP & MACHINERY CO.
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Minimum Trouble and Expense When Breaks Come

THE Smith Two-Part Standpipe Hydrant was designed to make the expense as small as possible when there is a break as the result, for instance, of careless auto truck driving.

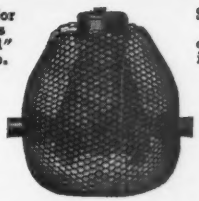
The upper section will always break first. It can easily be renewed. And of course there can be no leakage of water because the main valve is of the compression type.

SMITH
2-Part Standpipe Hydrant

THE A. P. SMITH MFG. COMPANY
EAST ORANGE, N. J.

THE FORD METER MITTEN
WILL SAVE FROZEN METERS

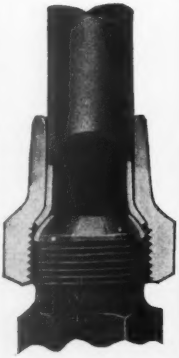
Made for meters up to 1" in size.



Simple and easy to install.

The Meter Mitten is a convenient and effective insulation jacket for basement meters. Conserves warmth in meter and water and prevents many meters from freezing. Costs but a fraction of the expense of a frozen meter. Write for further information.

The Ford Meter Box Co.
WABASH, INDIANA



HAYS begins at the MAIN

Since 1869 Hays has built to the exacting requirements of the Water Works Industry. Tap the Main with a Hays Tapping Machine and follow through with dependable Hays Service Fittings for iron pipe, lead or copper. Hays Corporation Stops and Curb Stops are made in all styles and sizes. Write for Catalog.

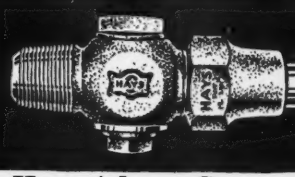
HAYS RADIUS SEAL JOINT

This type of joint was designed to meet the specific requirements of certain Water Departments and is available directly on all styles of Hays Copper Service fittings.

HAYS DOUBLE SEAL JOINT

This joint features a 2-faced pipe flare (45° + 90°) and a 2-faced machined seal (45° + 90°) which is by test the strongest flanged connection available. Available by Adapter on any Corporation Stop and directly on more than 400 styles and sizes of Hays Service and Plumbing fittings.

USE THE HAYS COPPER SERVICE METHOD



Hays Adapts Copper to meet any Requirements

As a pioneer in the development of copper pipe services, Hays has popularized its use by making it easily adaptable to meet existing line equipment economically. Every community can have the advantage that copper pipe makes possible from Main to Curb to House with the Hays Copper Service Method. Let us have your problem.

HAYS MFG. CO.

ERIE, PA.

HAYS

PLUMBING BRASS
BATH FIXTURES
COPPER PLUMBING
GAS SERVICE GOODS
WATER SERVICE GOODS
IRON PRODUCTS
TAPPING MACHINES

A PRETTY SCENE . . . (*for the Artist*)
but ANOTHER SOURCE *of* TROUBLE . . (*for the Plant Operator*)

ANOTHER source of worry to the Plant Operator is the high color caused by falling leaves in the raw water supply and during their decomposition the creation of a distinct taste and odor problem.

During the Fall period, rains flush out the swampy areas where stagnant water during the summer period has had an opportunity of leaching out the color from the vegetation. This contributes heavily to the color removal problem and to the accompanying intensified taste.

The solution of these problems lies in the application of increased coagulation dosage for color removal and NUCHAR for taste and odor removal.

The cost of furnishing palatable drinking water through the use of Activated Carbon is so negligible that every purification plant should consider its continuous use throughout the year.

Make your plans now to attend the Annual Convention of the Southwest Section of the American Water Works Ass'n to be held October 18-21, 1937, at the Hotel Galvez, Galveston, Texas.

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Centrifugal Water Works Pumps

Showing Automatic Switchgear Equipment

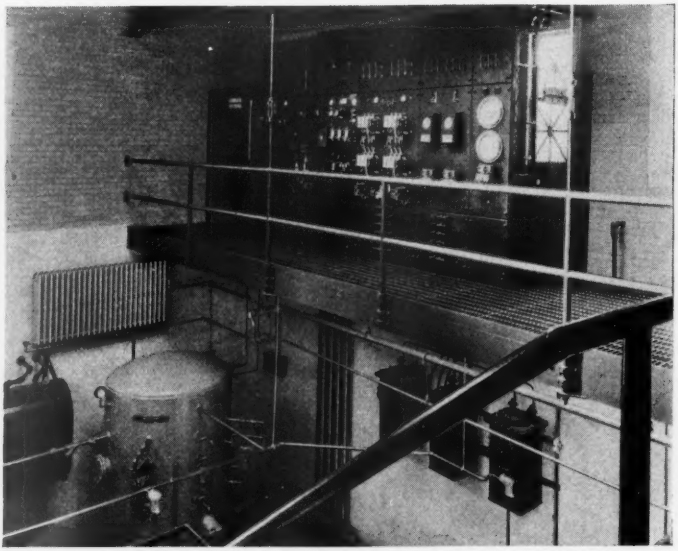


Illustration Shows Complete Switchboard
in the Ellis Water Treatment Plant at Norwood, Massachusetts

The main pumps in this station were illustrated in last issue.

This electrical apparatus automatically operates both high and low lift pumps according to predetermined liquid levels in the plant. It also controls chemical feed and mixing machinery in series with the low-lift pump supply.

Main pumps, vacuum pumps, float switches, etc., are in duplicate so that a second unit is automatically brought into service upon failure of the leading unit to perform.

We welcome your inquiries for particulars in regard to our systems.

TURBINE EQUIPMENT COMPANY

OF NEW ENGLAND

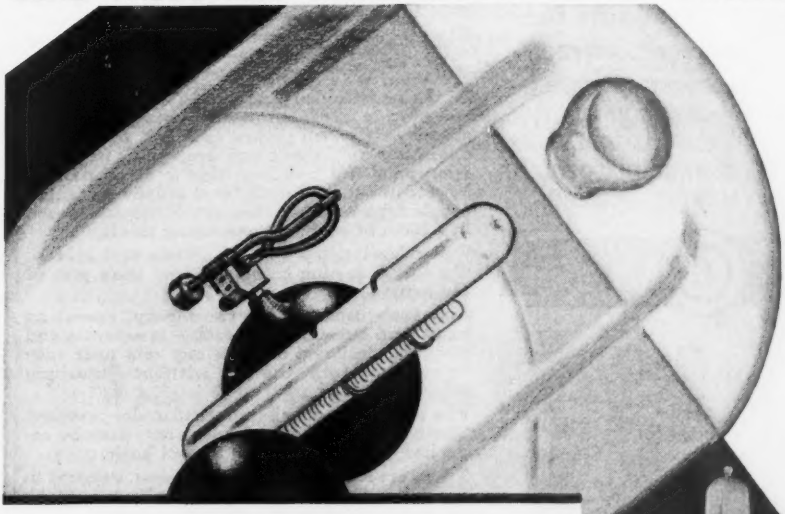
80 Federal Street Phone LIB erty 5993-5994 Boston, Mass.

New England Representative for

DE LAVAL STEAM TURBINE COMPANY

TRENTON, N. J.

REAL VALUE FOR YOUR CHLORINATION DOLLAR



GLASS HOUSES

are indispensable to chlorination

TROUBLE in a Visible Vacuum Chlorinator doesn't have a chance to develop into anything serious because the working parts are under the glass bell jar, always in plain sight.

But this is only one reason for Visible Vacuum Control. Visibility has an important corollary—Accessibility. When adjustments are needed—just drain the tray, lift the bell jar and the complete control mechanism is accessible for cleaning, repair or adjustment.

This important principle of combining Visibility and Accessibility, reinforced by standardized, high quality manufacturing and careful selection of the best materials has made the name "A W&T Visible Vacuum Chlorinator" synonymous with dependability, accuracy and low maintenance expense for over fifteen years.

When you are faced with the selection of equipment for the control of chlorination, visit a W&T Visible Vacuum installation. There you will learn why the Engineer, Superintendent and Public Health Official alike agree that Visible Vacuum Control is a real assurance for full value on your Chlorination dollar. Ask for technical publications 38, 157 and 158 describing W&T Visible Vacuum Chlorinators.

WALLACE & TIERNAN CO. INC.

Manufacturers of Chlorine and Ammonia Control Apparatus

NEWARK, N. J. Branches in Principal Cities

"The Only Safe Water is a Sterilized Water"

CHLORINATORS FOR WATER WORKS • SEWAGE PLANTS • SWIMMING POOLS
INDUSTRIAL PLANTS

SA-25

. . . and now the World's Best Cone Valves

Cone Valve Is Available for—

ALTITUDE CONTROL
PRESSURE REDUCING
CHECK VALVE
SLUDGE LINES
SEWAGE DISPOSAL
MANUAL CONTROL

WORLD'S BEST AUTOMATIC Valves

For more than 30 years Golden-Anderson has been known as the manufacturer of the WORLD'S BEST AUTOMATIC valves.

Now, Golden-Anderson offers you the supreme development in the Thru-Flow or Cone type Valves.

In the new Golden-Anderson Thru-Flow Valve the plug is unseated axially, rotated, and re-seated axially, all in one free and frictionless operating movement and drop tight in either open or closed position; this is accomplished by means of a simple and sturdy patented toggle type rotor mechanism which is encased in a bath of oil in the operating head.

The valve itself is just as simple and sturdy, the full flow plug being stronger than pipe of the same area.

And each unit — plug, valve body, operating head, and automatic control — is separate and distinct, designed so that any one unit may be removed from the line without disturbing other units.

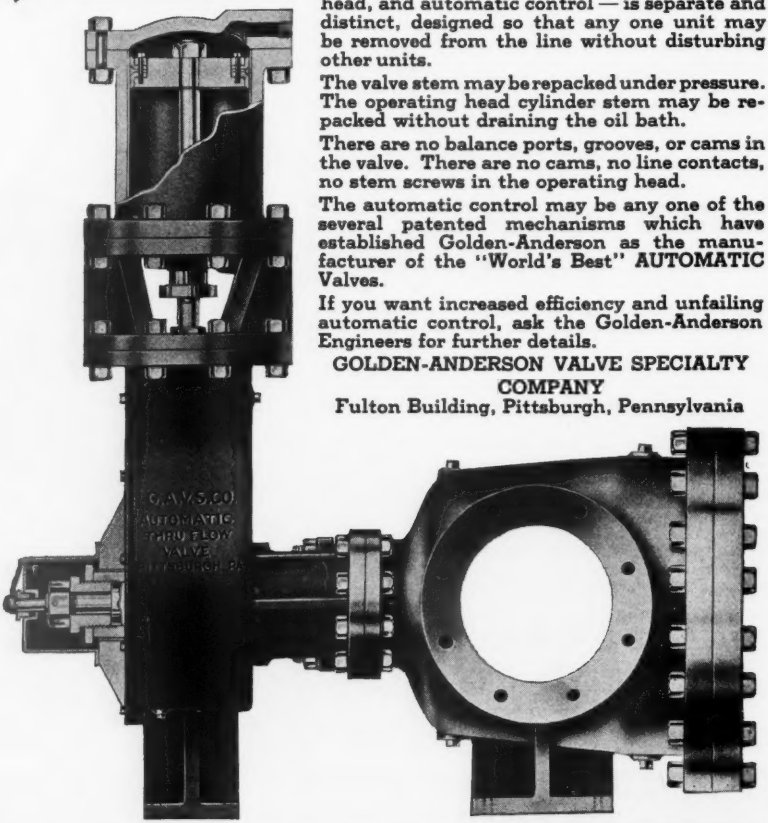
The valve stem may be repacked under pressure. The operating head cylinder stem may be re-packed without draining the oil bath.

There are no balance ports, grooves, or cams in the valve. There are no cams, no line contacts, no stem screws in the operating head.

The automatic control may be any one of the several patented mechanisms which have established Golden-Anderson as the manufacturer of the "World's Best" AUTOMATIC Valves.

If you want increased efficiency and unfailing automatic control, ask the Golden-Anderson Engineers for further details.

GOLDEN-ANDERSON VALVE SPECIALTY COMPANY
Fulton Building, Pittsburgh, Pennsylvania



Few Products In Any Field Are So Clearly Recognized for Superiority of Service As

SPUN **BITUMASTIC** LINING
REG. U.S. PAT. OFF.

The Report of the Committee on **PIPE LINE COEFFICIENTS** of the New England Water Works Association definitely proves the absolute economic necessity of lining all water pipe.

BITUMASTIC SPUN LINING is assigned a higher value of the coefficient *C* than any other lining. This insures the highest initial flow. More important —this high flow will be maintained for many years.

BITUMASTIC ENAMEL is not affected by tuberculating waters. Actual inspection of pipe in service and carrying tuberculating waters shows **BITUMASTIC** unaffected. This is real proof.

No taste, color, or odor imparted to the water.

Wailes Dove-Hermiston Corporation

General Office
17 Battery Place
New York

New England Representative
W. D. Barker, Room 606
250 Stuart St., Boston, Mass.

The "COREY" Hydrant

Why you should use GENUINE COREY HYDRANTS

Water to Nozzles quicker than other Hydrants.

Standpipe can be removed and replaced without shutting off water in main.

Ease of operation insured by knuckle joint principle.

Only 13 turns to completely open or completely close, but full flow with only 4 turns.

Nozzles screwed in — not leaded.

Can be lengthened without shutting off water.

Will not flood if broken.

Positive Drip — cannot freeze.

Standpipe can be revolved quarter turns — Nozzles always in right direction.

Ask for our Catalog "G"

CHARLES L. BROWN,
New England Sales Manager, Lock Box 2, Northboro, Mass.

RENSSELAER VALVE COMPANY
TROY, N. Y.

DONALDSON IRON CO.

MANUFACTURERS
...OF...

CAST IRON PIPE

...AND...

Special Castings for Water and Gas.
Also Flange Pipe and Fittings.

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WATER PROBLEM SOLVED BY NEEDHAM, MASS.

X. Henry Goodnough, Inc.
Engineers

● **The town of Needham can well be proud of its new water pumping station, for as city officials state, "Our water troubles will be over for a good many years to come." Believed to be the first Municipal Diesel driven Turbine Pump installation in New England, this system is equipped with a Fairbanks-Morse single-cylinder Diesel which powers an F-M Turbine Pump to economically supply the best quality water from an underground reservoir.**

Needham, as well as many other New England municipalities equipped by Fairbanks-Morse, is showing a substantial saving in operating costs. Get the facts on F-M power and pumping epuiqment by writing to Fairbanks, Morse & Co., 178 Atlantic Avenue, Boston, Mass.

FAIRBANKS-MORSE COMPANY

Power • Pumping • and • Weighing • Equipment

The
METER-MASTER
Increase Your Revenue
Cut Your Capital Investment
"Weed Out Oversized" Meters
The only rate recorder which can be attached to *any* make of meter, single or compound, without interrupting its record.

F. S. BRAINARD & CO.
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BRAINARD & HATCH
246 Palm Street
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CEMENT LINED SERVICE PIPE
Cement lined pipe has eliminated corrosion and metal contamination for 60 years.
Write for Literature
CEMENT LINED PIPE CO.
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PIERCE-PERRY CO.
Wholesalers of
Water Works Brass Goods.
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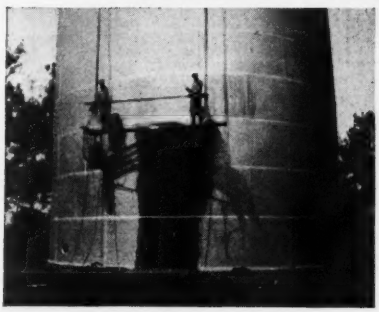
Valve and Service Boxes.
236 Congress St., Boston, Mass.
Telephone, Hancock 7817-7818.

Why take LESS than these Advantages in your BELL and SPIGOT Jointing Compound?

● Only in MINERALEAD do you get the INGOT form, so easy to store and handle—*plus* Assurance against change in composition en route to the job—*plus* Safety against rain and flood—*plus* No Delays when you add more to the melting pot.
● To these advantages add joints permanently tight and with ample reserve strength to stand heavy vibration.
● Write for folder and further information.

MINERALEAD

The ATLAS MINERAL Products Company *of Penna.*
MERTZTOWN, PENNSYLVANIA



STANDPIPE PAINTING
Sandblasting Power Chipping
Power Wire Brushing
PAINTING by BRUSH or SPRAY

J. H. TREDENNICK, Inc.
10 High Street Boston, Mass.

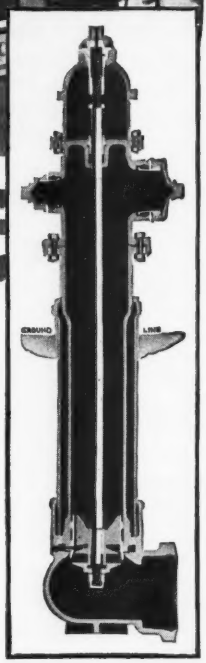
"THOSE TWO MEN HAVE OVERHAULED ALL OUR OLD

MATHEWS HYDRANTS

IN SPARE TIME"

"This spring we wanted an appropriation to overhaul some 50-year-old hydrants. It turned out we only needed to replace some packing, a few valve-leathers and a few worn operating nuts. For we have Mathews Hydrants.

"Those two men go out to a hydrant, unscrew the barrel, lift it out and put in our spare. Then they take the old one back to the shop, check it all over, repack and lubricate it in spare time, and use it to replace another. They've about covered the town now, they've had every hydrant in the shop, they haven't dug a shovelful of dirt, and we didn't need an appropriation."



The inside of the elbow is threaded. The entire barrel including all working parts can be lifted out for inspection, overhaul, or accident-repair, without digging.

SPECIFY MATHEWS

You'll save money for the next 75 years.

R. D. WOOD CO.

400 Chestnut St., Phila., Pa.

manufacturers of SAND-SPUN PIPE which is centrifugally cast in sand molds, and R. D. Wood heavy-duty gate valves for water works.

Mathews Hydrants

BUILT FOR EMERGENCIES

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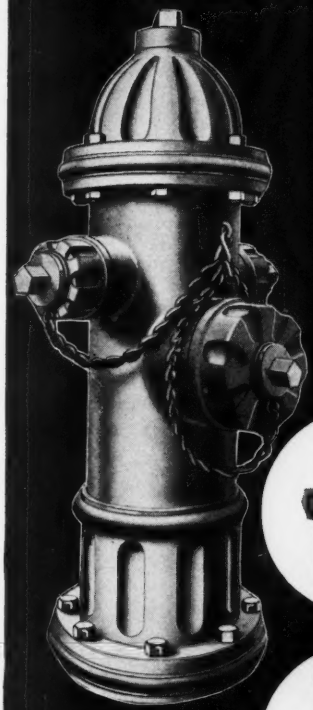

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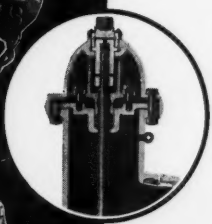

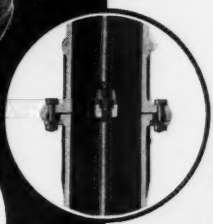

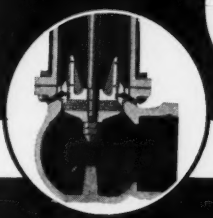

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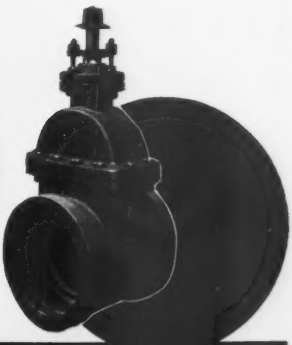

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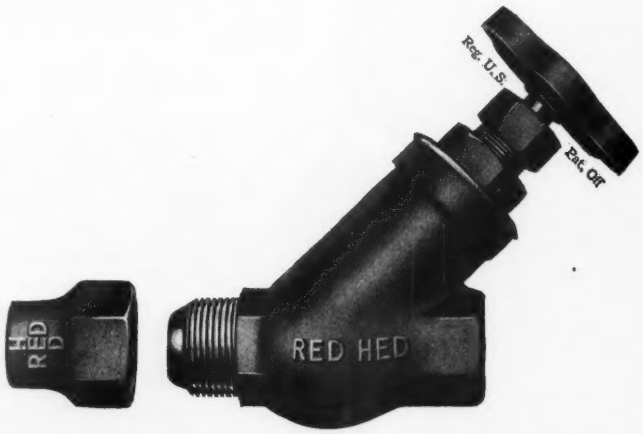

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Parallel Seat

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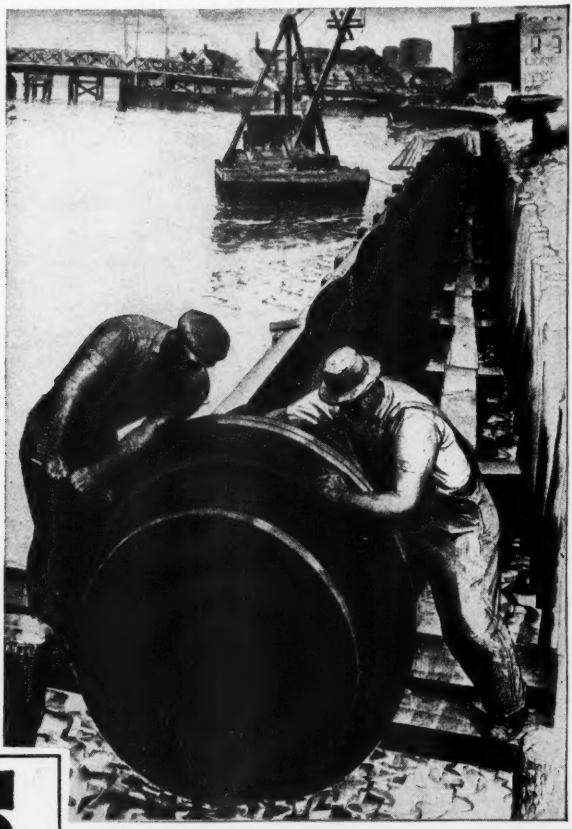

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